DECEMBER 1960
No. 6
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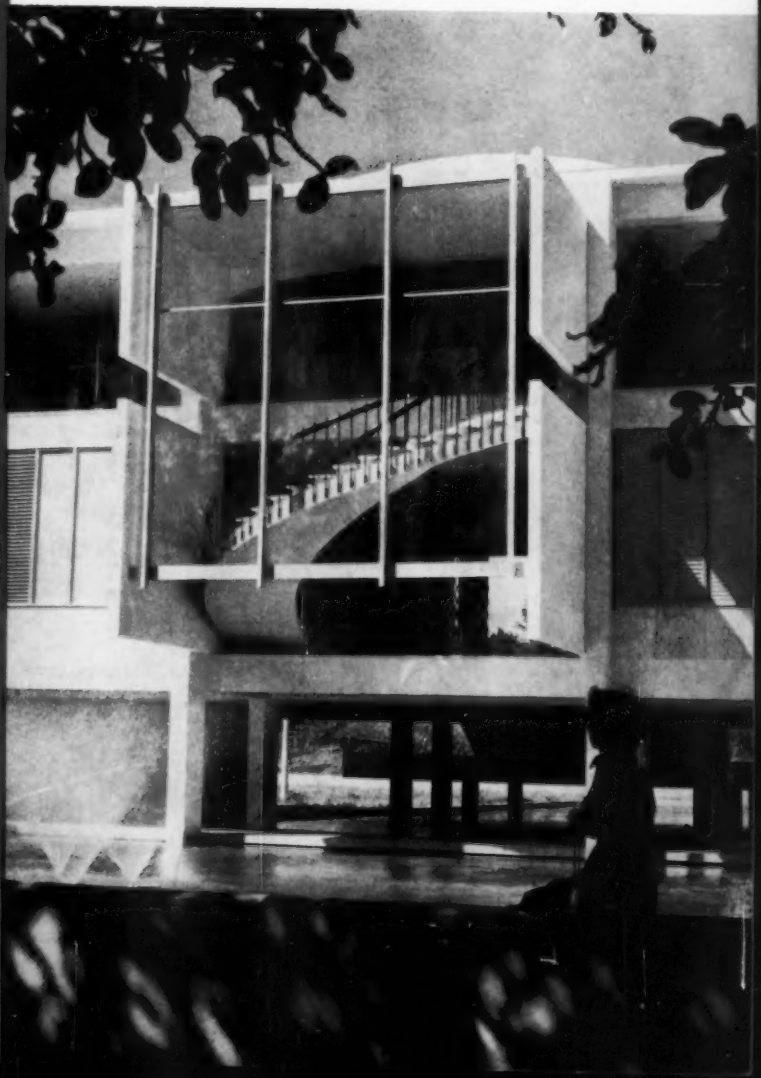

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ALAN H. MATTOCK and LADISLAV B. KRIZ present results of their research on "Ultimate Strength of Nonrectangular Structural Concrete Members."

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KOLBJORN SAETHER reveals the role of "The Structural Membrane" in replacing the shape of deflected elastic membranes of thin shell structures.

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Comprehensive laboratory data on "Freezing and Thawing Tests of Lightweight Aggregate Concrete" is reported by PAUL KLIEGER and J. A. HANSON.

•

Coauthors BRYANT MATHER and WILLIAM O. TYNES describe an "Investigation of Compressive Strength of Molded Cylinders and Drilled Cores of Concrete."

•

RESEARCH COMMITTEE of ACI COMMITTEE 714, Concrete Bins and Silos, offers a report on "An 11-Year Study of Concrete Stave Silo Durability."

•

F. E. LEGG, JR. and W. W. MCLAUGHLIN discuss "Gravel Beneficiation in Michigan."

On the Cover—Helicoidal central staircase in the Supreme Court Building, San Juan, Puerto Rico. At the head of this reinforced concrete staircase is the only circular Supreme Courtroom in the world. Details of the design and construction—central angle: 220 deg; radii: $R_i = 5$ ft 6 in., $R_o = 10$ ft 10 in.; slab thickness (normal to slab) = 10 in., reinforcement: three 1-in. diameter bars each side (one top, one bottom, one middle), other top bars, four 5/8-in. diameter, other bottom bars 19—5/8-in. diameter all continuous; ties: 5/8-in. diameter at 4 in. on centers at supports increasing to 12 in. at midspan. Toro-Ferrer, Puerto Rico, were the architects. Consulting architect was Charles H. Warner, Jr., New York. Harold E. Leeds was design consultant—Photo Puerto Rico News Service.

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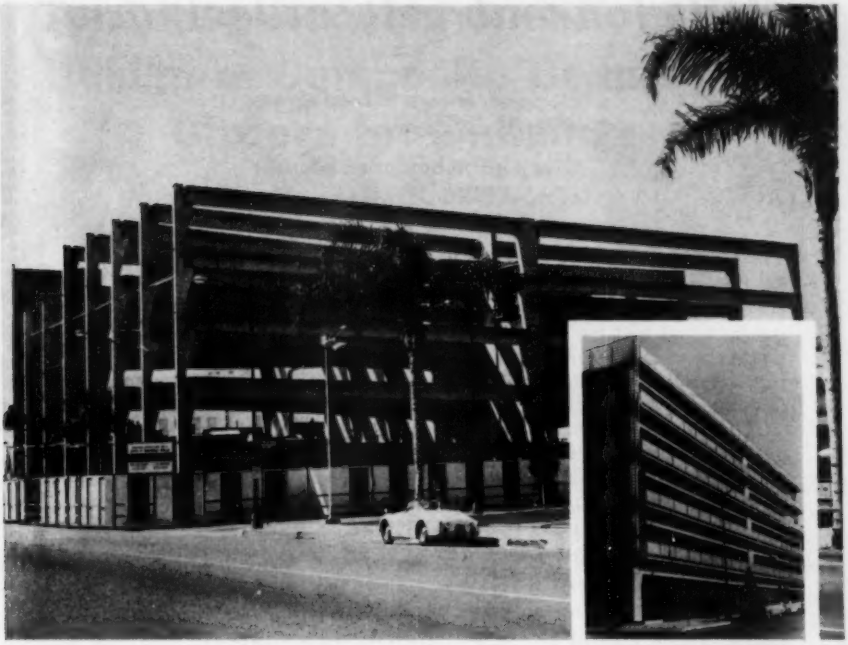

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Tentative Recommendations for Design of Composite Beams and Girders for Buildings

Reported by ACI-ASCE COMMITTEE 333

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ACI-ASCE Committee 333 was organized in 1956 to prepare recommendations for the design and construction of structures composed of prefabricated beams combined with cast-in-place slabs. After a review of the existing information and practices, the committee has channeled one part of its activities toward preparation of recommendations for the design of composite beams and girders for buildings. The results of this work are reported herein. The committee expects to prepare further reports after the completion of research investigations now in progress.

The progress report is written in two parts. Tentative design recommendations are presented in the first part. The second part contains explanations of the provisions of the design recommendations.

CHAPTER 1 — GENERAL PROVISIONS

101—Definition

Composite beams and girders are comprised of prefabricated beams and of a cast-in-place reinforced concrete slab so interconnected that the component elements act together as a unit.

102—Scope

These recommendations apply to buildings subjected primarily to static loads. Structures containing prefabricated beams made of precast reinforced concrete, precast prestressed concrete, or either rolled or built-up steel-sections are included.*

The direction and coordination of the efforts leading to this progress report were assigned to Philip P. Page, Jr. The committee wishes to acknowledge his efforts, which were primarily responsible for the prompt completion of the report.

*Guides for the design of timber-concrete slabs and T-beams may be found in technical literature; see, for example, *Timber Design and Construction Handbook*, F. W. Dodge Corp., New York, 1956, pp. 214-229.

103 Design of slab

The slab may be designed as either a one-way or two-way slab in accordance with "Building Code Requirements for Reinforced Concrete (ACI 318-56)." In the design of the slab, the stresses caused by composite action may be neglected.

104—Design of prefabricated beams

104.1 Precast reinforced concrete beams — Reinforced concrete beams should be designed according to "Building Code Requirements for Reinforced Concrete (ACI 318-56)" except as otherwise stated in Chapter 2 of these recommendations.

104.2 Precast prestressed concrete beams — Prestressed concrete beams should be designed according to "Tentative Recommendations for Prestressed Concrete" (ACI-ASCE Committee 323, Jan. 1958)[1] except as otherwise stated in Chapter 3 of these recommendations.

104.3 Steel beams — Steel beams should be designed according to "Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings" (AISC 1949) except as otherwise stated in Chapter 4 of these recommendations.*

105—Effective width of flange

105.1 Flange on both sides of beam — For composite T-beams having the slab on both sides of the prefabricated beam, the effective width of the concrete flange should not exceed one-fourth of the span length of the beam, and the overhanging width on either side of the prefabricated beam should not exceed eight times the thickness of the slab nor one-half the clear distance to the next beam.

105.2 Flange on one side of beam — For beams having the slab on one side only, the effective overhanging flange width should not exceed 1/12 of the span length of the beam, nor six times the slab thickness nor one-half the clear distance to the next beam.

106—Mixed construction

The use of noncomposite beams in systems using composite beams is permissible provided that it is consistent with the deformational characteristics of the structure.

107—Deflections

107.1 Live load deflections — Live load deflections should be computed on the basis of the moment of inertia of the transformed composite section using the full value of the modulus of elasticity E_c.

107.2 Dead load deflections

1. For beams shored during construction, the dead load deflections should be computed on the basis of the moment of inertia of the trans-

*The use of plastic design for composite construction will be the subject for future recommendations of ACI-ASCE Committee 333.

formed composite section using one-half the value of the modulus of elasticity E_c.

2. For beams not shored during construction, the dead load deflections should be computed on the basis of the moment of inertia of the prefabricated beam alone except that deflections due to dead loads applied after the concrete slab has attained 75 percent of the specified 28-day strength should be computed according to Section 107.2.1.

108—Continuity

108.1 Determination of moments, shears, and thrusts — Moments, shears, and thrusts produced by external loads should be determined by elastic analysis. For the purposes of such analysis, the moment of inertia of the gross composite section may be used throughout the length of the beam.

108.2 Sections resisting negative moments

1. In negative moment regions of continuous or cantilever beams, the bending moment may be assigned either to the prefabricated section alone or to the composite section composed of the prefabricated section and the slab reinforcement. Such assignment shall be consistent with the design of shear connection (Section 108.3).

2. When the slab is continuous at the supports of beams, reinforcement should be provided sufficient to prevent excessive cracking of the slab.

108.3 Shear connection in regions of negative moments

1. When the negative moments are assigned to the prefabricated section alone, shear connection between the prefabricated beam and the slab need not be provided in the regions of negative moments.

2. When the negative moments are assigned to the composite section, shear connection must be provided throughout the full length of the beam.

109—Construction methods

109.1 Shoring — When shores are used, they should be kept in place until the cast-in-place concrete has attained 75 percent of the specified 28-day strength.

109.2 Camber — Necessary provisions should be taken in the design and construction to prevent excessive dishing of the slab in beams built with shores and excessive thickening of the slab in beams built without shores.

109.3 Treatment of beam surfaces — Surfaces of prefabricated beams in contact with the slab should be cleaned of any foreign or loose material before casting the slab. It is preferable to leave the contact surfaces of steel beams unpainted.

CHAPTER 2 — SLAB ON PRECAST REINFORCED CONCRETE BEAMS

201—Allowable stresses, moduli of elasticity, and load factors

201.1 Allowable stresses — Allowable stresses for reinforcing steel and concrete specified in "Building Code Requirements for Reinforced Concrete (ACI 318-56)" are recommended for the design of composite beams. In structures composed of elements with different concrete strength, the allowable stresses in each portion should be governed by the concrete strength of the portion under consideration.

201.2 Tension in concrete — The tensile resistance of concrete should be neglected.

201.3 Moduli of elasticity — The following values for the moduli of elasticity should be used:

1. Steel: $E_s = 30{,}000{,}000$ psi
2. Concrete: $E_c = 1000\ f_c'$ where f_c' is the 28-day compressive strength of the concrete under consideration.

201.4 Load factors — For designs based on Section 203, load factors given in the Appendix to "Building Code Requirements for Reinforced Concrete (ACI 318-56)" are recommended.

202—Determination of flexural stresses

202.1 Design method — The design of reinforced concrete members may be made with reference to allowable stresses, working loads, and the accepted straight line theory of flexure except in designs based on Section 203.

202.2 Loading conditions

1. For unshored construction the dead load of the precast beam and all other loads applied prior to the concrete slab attaining 75 percent of its specified 28-day strength should be assumed as carried by the precast beam alone. Live loads and dead loads applied after the concrete has attained 75 percent of its specified 28-day strength should be assumed as carried by the composite section.

2. For adequately shored construction all loads should be assumed as carried by the composite section.

202.3 Deformational stresses — Deformational stresses, including the effects of creep, shrinkage, and temperature, need not be considered except in unusual cases.

203—Determination of ultimate flexural strength

203.1 Design method — Ultimate strength of a composite section may be computed in the same manner as the ultimate strength of an integral member of the same shape following the procedure outlined in the Appendix to "Building Code Requirements for Reinforced Concrete (ACI 318-56)." In computing the ultimate strength of a section, no distinction should be made between shored and unshored beams.

203.2 Limitations — For beams designed on the basis of ultimate strength and built without shores, the effective depth of the composite section used in the computation of the ultimate moment should not exceed:

$$d_e = \left(1.15 + 0.25\,\frac{M_L}{M_D}\right) d_p$$

where d_e = effective depth of the tension reinforcement in the composite section
M_L = moment produced by live load and superimposed dead load
M_D = moment produced by dead load prior to the concrete attaining 75 percent of specified 28-day strength
d_p = effective depth of the tension reinforcement in the precast section

When the specified yield point of the tension reinforcement exceeds 40,000 psi, beams designed on the basis of ultimate strength should always be built with shores unless provisions are made to prevent excessive tensile cracking.

203.3 Construction loads — The precast beam alone should be investigated to assure that the loads applied before the concrete has attained 75 percent of its specified 28-day strength do not cause moment in excess of 1/1.8 times the ultimate moment capacity of the precast section.

204—Determination of shear

204.1 Reinforcement of the web — Web reinforcement should be designed in the same manner as for an integral T-beam of the same shape. All stirrups should be extended into the cast-in-place slabs.

204.2 Horizontal shear

1. The shear at any point along the contact surface may be computed as:

$$v = \frac{VQ}{I}$$

where v = horizontal shear per unit length of beam
V = total external shear at the section considered caused by both live and dead loads
Q = statical moment of the transformed area on one side of the contact surface about the neutral axis of the composite section
I = moment of inertia of the transformed composite section neglecting the tensile resistance of concrete

2. If the horizontal shear, v, exceeds the capacity of bond as recommended in Section 205.3, shear keys should be provided throughout the length of the member. Keys should be proportioned according to the concrete strength of each component of the composite member.

205—Shear connection

205.1 Shear along contact surface — Shear may be transferred along the contact surface by bond or shear keys. It should be assumed in the design that the entire shear is transferred either by bond or by shear keys.

205.2 Vertical ties — Mechanical anchorage in the form of vertical ties to prevent separation of the component elements in the direction normal to the contact surfaces should be provided. Spacing of such ties should not exceed four times the thickness of the slab nor 24 in. A minimum cross-sectional area of the ties in each foot of span of 0.15 percent of the contact area but not less than 0.20 sq in. is recommended. It is preferable to provide all ties in the form of extended stirrups.

205.3 Capacity of bond

1. The following values are recommended for the allowable bond stress at the contact surfaces:

(a) When minimum steel tie requirements of Section 205.2 are followed and the contact surface of the precast element is smooth (a smooth surface is one which has been cast against a form, troweled, or floated ..... 40 psi

(b) When minimum steel tie requirements of Section 205.2 are followed and the contact surface on the precast element is rough ..... 160 psi

(c) When additional vertical ties are used, the allowable bond stress on a rough surface may be increased at the rate of 75 psi for each additional area of steel ties equal to 1 percent of the contact area.

2. The capacity of bond at ultimate load may be taken as twice the values recommended in Section 205.3.1.

CHAPTER 3 — SLAB ON PRECAST PRESTRESSED CONCRETE BEAMS

301—General

Composite structures consisting of precast or cast-in-place slabs resting on prestressed concrete beams may be designed in accordance with "Tentative Recommendations for Prestressed Concrete" (ACI-ASCE Committee 323, Jan. 1958),[1] except that it is recommended to design the shear connection according to the provisions of Sections 204 and 205 of these recommendations.

CHAPTER 4 — SLAB ON STEEL BEAMS

401—Allowable stresses and moduli of elasticity

401.1 Allowable stresses

1. The allowable stresses for steel, except for reinforcing bars, specified in "Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings" (AISC 1949) are recommended for the design of composite beams.

2. The allowable stresses for concrete and for reinforcing bars specified in "Building Code Requirements for Reinforced Concrete (ACI 318-56)" are recommended for the design of composite beams.

401.2 Tension in concrete — The tensile resistance of concrete should be neglected.

401.3 Moduli of elasticity — The following values of the moduli of elasticity should be used:

1. Steel: $E_s = 30{,}000{,}000$ psi
2. Concrete: $E_c = 1000\ f_c'$ where f_c' is the 28-day compressive strength of the concrete

402—Determination of flexural stresses

402.1 Design method

1. Flexural stresses should be determined at the working load level on the basis of the moment of inertia of the transformed composite section.

2. The transformed area of the composite section should be computed on the basis of the modular ratio $n = E_s/E_c$.

3. For beams built without temporary supports and designed according to Section 402.2, the section modulus of the composite section used in computations should not exceed the value:

$$S_c = \left(1.35 + 0.35\,\frac{M_L}{M_D}\right) S_s$$

where S_c = section modulus of the tension flange of the composite beam
M_L = moment produced by live load and superimposed dead load
M_D = moment produced by dead load prior to the concrete attaining 75 percent of specified 28-day strength
S_s = section modulus of the tension flange of the steel beam alone

4. The steel beam alone should be investigated to assure that the actual stresses in the steel do not exceed the allowable values for steel (Section 401.1.1) before the concrete has attained 75 percent of its specified 28-day strength.

402.2 Loading conditions — It may be assumed in the stress computations (except for provision of Section 402.1.4) that all dead loads and live loads are resisted by the composite section whether or not temporary supports are used.

402.3 Deformational stresses — Deformational stresses, including the effects of creep, shrinkage, and temperature, need not be considered except in unusual cases.

403—Shear

403.1 Web stresses — The web of the steel beam should be capable of carrying the entire external shear without exceeding the allowable shearing stress.

403.2 Horizontal shear between slab and beam

1. The horizontal shear at the junction of the steel beam and the concrete slab or haunch should be computed as follows:

$$v = \frac{VQ}{I}$$

where v = horizontal shear per unit length of the beam
V = total external shear at the section considered caused by dead and live loads
Q = statical moment of the transformed area on one side of the contact surface about the neutral axis of the composite section
I = moment of inertia of the transformed composite section

In computing the section properties, the modulus of elasticity of concrete E_c given by Section 401.3.2 should be used.

2. The horizontal shear should be assumed to be transferred entirely by shear connectors except as noted in Section 403.2.3.

3. In beams fully encased in concrete, the entire horizontal shear may be assumed to be transferred by bond and friction provided that such beams are designed in accordance with Section 13 of "Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings" (AISC 1949). No shear connectors or ties are required in such beams.

403.3 Concrete haunch — The horizontal shearing stresses in the concrete haunch between the steel beam and the concrete slab should not exceed the allowable shear stress for concrete. If the allowable shear stress is exceeded, the haunch should be provided with web reinforcement. The shear connectors may be considered as web reinforcement.

404—Shear connectors and ties

404.1 Allowable loads for connectors — The shear connectors should be designed on the basis of an allowable load given as follows:

1. Stud shear connectors:

$$q = 165\, d^2 \sqrt{f_c'} \text{ for } h/d \text{ equal or larger than } 4.2$$

$$q = 40\, hd \sqrt{f_c'} \text{ for } h/d \text{ smaller than } 4.2$$

where q = allowable load per one stud, lb
h = height of stud, in.
d = diameter of stud, in.
f_c' = 28-day compressive strength of concrete, psi

2. Spiral shear connectors:

$$q = 1900\, d\, \sqrt[4]{f_c'}$$

where q = allowable load per one pitch of spiral, lb
d = diameter of bar, in.

3. Channel shear connectors:

$$q = 90\,(h + 0.5\,t)\, w \sqrt{f_c'}$$

where q = allowable load per one channel, lb
w = length of channel, in.
t = thickness of web, in.
h = maximum thickness of flange, in.

4. For connectors other than the above, the allowable load should be developed from test data.

404.2 Spacing and cover of connectors

1. Shear connectors should be spaced along the beam in accordance with the formula

$$s = \frac{q}{v}$$

where s = spacing of shear connectors
q = allowable load per shear connector or on a group of connectors placed at the same section
v = horizontal shear per unit length of beam defined in Section 403.2.1

2. Shear connectors should have at least 1 in. concrete cover in all directions.

404.3 Ties

1. Mechanical ties should be provided between the slab and the beams to prevent separation. Such ties may be a part of the shear connectors.

2. The maximum spacing of ties should not exceed 24 in.

Explanations of Tentative Recommendations

The ACI-ASCE committee on composite construction has prepared tentative recommendations for the design of composite beams and girders in buildings. Since no code is available for the design of composite buildings, the release of these *tentative* recommendations is considered desirable even though important research projects on composite beams are now in progress. As further data are developed from the research projects, the committee expects to make appropriate modifications in its future reports.

In producing these recommendations the committee was faced with the problem of being consistent with existing codes for the materials involved. It felt that it was not the province of this committee to rewrite accepted practice but rather to correlate and draw upon it producing new concepts only where no accepted practice previously existed.

Lastly, the committee felt that any recommendations produced because of the need for a usable guide should be written in terms familiar to most designers. Thus, the recommendations have been limited generally to working stress design, even though several provisions are based on, or refer directly to, the conditions at ultimate strength.

The recommendations are limited to structures with concrete made from conventional hard rock aggregates. No data were available to the committee on the behavior of composite beams made with lightweight aggregate concretes.

Composite action

To explain the reasoning behind the committee's recommendations, it will be necessary first to review the action of a simply supported composite beam. After the prefabricated member has been erected, its lower flange is subjected to a tension which can be computed by general flexural theories (Fig. 1). Immediately after casting, the slab which is still plastic adds no strength but merely dead load increasing the lower flange stresses. After the slab has hardened, it becomes the top flange of the composite section; additional loads causing further tension in the lower flange are resisted by the entire composite section. However, this tension is less per *unit* load because the composite section is stronger. Since a *unit* load applied before commencement of composite action creates greater flange stresses than one applied after, attempts have been made to place as much of the load as possible on the composite section rather than on the prefabricated beam alone. The classic method was to shore the beam.

The discussion so far has dealt with stresses in the elastic range; that is, with stresses below the yield point rather than at the ultimate. The recommendations of ACI-ASCE Committee 323,[1] dealing with precast prestressed stems and cast-in-place slabs, state that the ultimate strength of the composite member is the same as that of a monolithic T-beam of equal dimensions. The fact that the composite beam may be built with or without shores does not influence the ultimate strength.

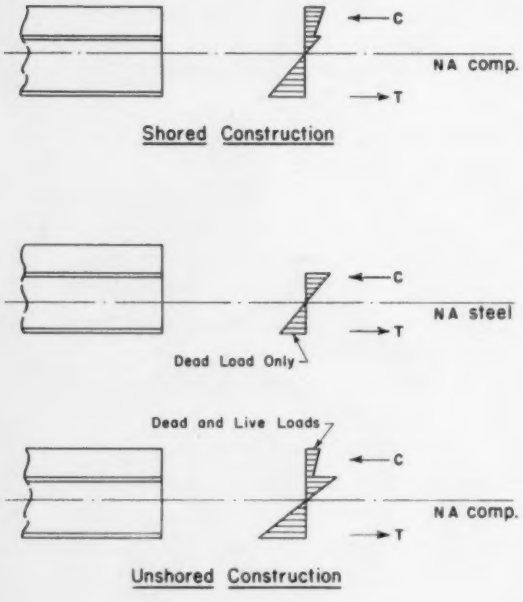

Fig. 1 — Stress distribution at working load

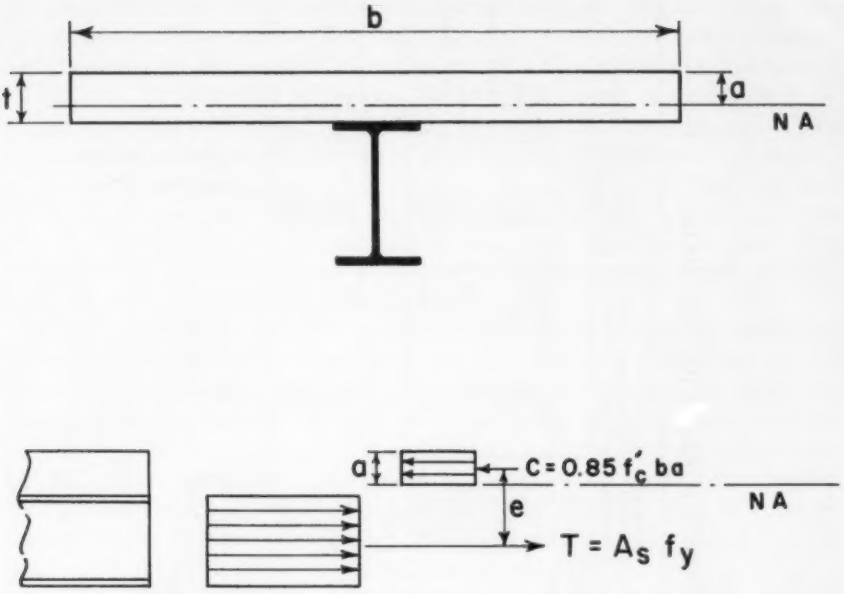

Fig. 2—Stress distribution at ultimate bending moment

A similar situation exists in a steel beam with a cast-in-place slab. Assuming that *at ultimate load* the neutral axis lies in the concrete slab, the stress distribution corresponding to the ultimate bending moment may be approximated as shown in Fig. 2.* The entire tensile force is carried by the steel beam stressed uniformly to its yield point. The entire compressive force is carried by the concrete above the neutral axis uniformly stressed to 85 percent of its cylinder strength. The location of the neutral axis is such that the total force in the concrete equals the total force in the steel and the ultimate bending moment is this total force times the distance between the centers of gravity of the two stress blocks. This fully plastic stress distribution is independent of the manner in which the stresses are induced into the beam.

Table 1 contains comparisons between computed and actual ultimate loads. Data for 15 beams with depths of steel section varying between 3 and 24 in. have been tabulated. The actual ultimate loads are those observed in the tests. The computed ultimate loads have been calculated from the reported properties of materials used in the tests. It may be noted that for all beams that failed in flexure the correlation between

*A similar theory can be developed for beams with the neutral axis below the top surface of the steel beam.

TABLE 1 — COMPARISON OF COMPUTED ULTIMATE LOADS WITH TEST DATA

Specimen	Reference	Depth of steel beam,* in.	Average† yield point, ksi	Size of slab,* in. x in.	Strength of concrete, ksi	Type of shear connector	Ultimate load‡			Mode of failure
							Computed, kips	Test, kips	Test / Computed	
Beams without shoring§										
Double T	2	18	37.9	131x6	3.93	stud	156.3	172.8	1.11	flexure
C	3	8	37.6	24x4.9	3.60	bar+loop	54.5	57.2	1.05	flexure
B24W	4	24	36.8	72x6.3	5.10	channel	116.9	111	0.95	did not fail
B21W	4	21	37.0	72x6.1	5.75	channel	96.0	79	0.82	horizontal shear
T1	5	3	37.4	18x1.8	5.10	channel	13.0	13.0	1.00	flexure
T2	5	3	37.4	18x1.7	1.62	channel	9.7**	10.2	1.05	flexure
T3	5	3	37.4	18x1.8	4.25	channel	12.2	8.7	0.71	horizontal shear
1	6	8	43.8	30x4.5	7.04	spiral	75.7	77.0	1.02	flexure
2	6	8	39.0	30x4.5	7.38	spiral	65.0	70.5	1.08	flexure
Beams with shoring or preloaded††										
D	3	8	37.9	24x4.9	3.30	bar+loop	45.8	44.1	0.96	flexure
E	3	8	38.0	24x4.8	3.61	bar+loop	54.5	54.9	1.01	flexure
B24S	4	24	36.1	72x6.2	5.67	channel	115.1	115	1.00	did not fail
B21S	4	21	37.6	72x6.3	6.53	channel	99.2	102	1.03	flexure
3	6	8	39.8	30x4.5	7.04	spiral	66.0	67.5	1.02	flexure
4	6	8	42.7	30x4.5	7.38	spiral	74.5	71.8	0.96	flexure

*Rounded-off values.
†Average weighted according to areas and corresponding yield points.
‡Dead load not included.
§Average test/computed for flexural failures:1.052.
**Neutral axis in the steel section; for all other specimens neutral axis in the slab.
††Average test/computed for flexural failures: 0.996.

the computed and observed ultimate loads is good: the difference between the two loads varies between −4 and +11 percent. The observed load is substantially smaller than the computed one only for Specimens B21W and T3 designed with extremely weak shear connections.

The specimens listed in Table 1 are divided into two groups: those built without temporary supports and those either shored or loaded with additional weights during construction. Using the same method of computation, the ultimate moment capacity was predicted with essentially equal accuracy for both shored and unshored beams. The ultimate flexural strength of a composite steel-concrete beam is independent of the construction method.

Based on a design stress of 20,000 psi and a yield stress of 33,000 psi, the factor of safety at first yielding of a noncomposite steel beam is 1.65. At ultimate strength the stress distribution is plastic and thus the factor of safety against failure is higher. For a symmetrical rolled section, the ultimate load is approximately 1.85 times the design load. On the other hand, for a composite beam designed on the assumption that the composite section takes all loads, the ultimate load is approximately 2.2 to 2.5 times the design load. The addition of a bottom cover plate does not change these values appreciably. If the composite beam were designed on the assumption that the steel beam alone resisted the dead loads, the ratio of ultimate to design load would be still higher.

As a result of these observations the committee makes the following recommendation:

> The design of composite beams made up of a prefabricated *steel* beam and a cast-in-place concrete slab may be based upon working stresses calculated by assigning all loads to the composite section regardless of construction method. That is, all loads may be considered in the design as resisted by the composite section even though shores are not employed during construction.

This recommendation is limited to composite sections with steel beams (Section 402). It is not recommended for the design of composite sections with either precast reinforced concrete beams or precast prestressed concrete beams.

For sections composed of a slab and a precast reinforced concrete beam, the committee recommends two alternate methods of design: a working stress design (Section 202) and an ultimate strength design (Section 203). The working stress procedure takes into account the construction method, while the ultimate strength procedure is independent of the method of construction. Both procedures follow principles well established in the design of reinforced concrete.

The ultimate strength of a composite concrete T-beam is independent of the method of construction. Thus, the load factors used in the ultimate strength design provide the necessary overload capacity regardless of

the type of construction used. The recommendation of the ultimate strength design as an alternate method for reinforced concrete sections is warranted both by the availability of information and by the familiarity of the designers with such design.

The need for a working stress procedure is self-evident. However, in contrast to composite sections with steel beams, the reserve strength of a composite concrete beam beyond first yielding of the reinforcement can be small. Therefore, it is recommended that the construction method in the working stress design of concrete beams be considered.

Finally, the committee considered satisfactory the currently available method for the design of composite sections made with prestressed concrete beams (Chapter 3).

Construction considerations

Although the procedure recommended for the design of sections with steel beams provides adequate safety against failure of the composite beam, it does not limit either the erection stresses or the actual stresses at working load. To guard against possible damage to the steel section, Sections 402.1.3 and 402.1.4 place limits on the total stress and on the stresses in the steel beam prior to hardening of the slab.

In beams built with temporary supports the actual stresses exceed the computed values only as a result of volume changes of concrete. The increases in the governing tension in the steel section resulting from volume changes are usually small and may be safely neglected. On the other hand, in beams built without temporary supports, the actual stress may be substantially in excess of the computed value because the dead load is in reality resisted by the steel section alone, while the recommended procedure assigns it to the composite section. The limit on the section modulus of the composite beam (Section 402.1.3) places a maximum of 27,000 psi on the total stress caused by dead and live loads in an unshored beam. This provides safety against yielding comparable to that of a noncomposite beam.

To avoid damage to the steel section during construction, it is recommended in Section 402.1.4 to limit the stresses in the bare steel beam before hardening of the slab to the conventional allowable values for steel. This provision is particularly needed for unsymmetrical steel sections with light compression flanges.

The ultimate strength procedure for the design of sections with precast reinforced concrete beams provides adequate safety against failure of the composite beam but additional limitations are needed to guard against damage to the precast beam during erection and against excessive widening of the tension cracks at working loads. The provisions of Sections 203.2 and 203.3 serve this purpose.

Except for small changes caused by differential shrinkage, the stresses in a composite beam fully shored during construction are the same

as in a cast-in-place monolithic beam. In beams built without temporary supports, the weight of the precast beam and of the concrete slab is resisted by the precast section alone. However, the ultimate strength of the composite section is the same regardless of the construction procedure. Thus actual stresses caused by dead and live loads are higher if shores are not used. The limitation on the effective depth of the composite beam (Section 203.2) limits the total stress caused by dead and live loads to 75 percent of the specified yield point f_y of the tension reinforcement.

The limit of 0.75 f_y not only keeps the stresses well below the yield point but also prevents excessive tensile cracking of the beam provided that the yield point is only moderately high. The committee recommends that, unless provisions are made to prevent excessive widening of tensile cracks, composite beams designed on the basis of ultimate strength with the yield point of the steel in excess of 40,000 psi should not be constructed without the use of shores. When the yield point does not exceed 40,000 psi, the actual stresses in the tension reinforcement in a composite beam built without temporary supports will be no higher than the maximum values attainable in monolithic beams designed by the ultimate strength design procedure recommended in "Building Code Requirements for Reinforced Concrete (ACI 318-56)."

The provision of Section 203.3 guards against damage to the precast section during construction.

Shear connection

It has been assumed tacitly in the preceding discussion that the shear connection between the prefabricated beam and the cast-in-place slab is capable of developing the ultimate moment of the composite section. It may be observed in Fig. 2 that at ultimate load the shear connection must develop the fully plastic force C (or T) through horizontal shear in the length between the sections of zero and maximum bending moments. If the connection is able to yield and still develop the fully plastic or ultimate strength of the beam, the exact distribution of horizontal shear is not critical. On the other hand, if the connection is brittle or if its yielding causes a significant loss of composite action, the distribution of this horizontal shearing force has to be considered. Experimental studies are now in progress which are expected to supply the information needed for formulating an ultimate strength procedure for the design of the shear connection.

Another approach to the design of the shear connection makes use of the elastic horizontal shear formula. Such approach, based on experimental observations, was developed for the design of composite bridges[7] and was adapted for this report. It provides a connection capable of developing the fully plastic moment capacity of the composite beam.

TABLE 2 — HORIZONTAL SHEAR FAILURES OF COMPOSITE CONCRETE-CONCRETE BEAMS

Specimen	Reference	Type of joint surface	Ties	a/d*	Horizontal shear strength, psi
BS-I	8	Smooth	#3— 6 in. on centers	3.0	350
BS-II	8	Smooth	#3—16 in. on centers	4.2	340
A2	9	Smooth	none	6.5	78
C2	9	Smooth	#4— 6 in. on centers	6.5	100
A3	9	Smooth	none	3.3	119
J	10	Smooth	none	8.0	122
BRS-I	8	Rough	#3— 6 in. on centers	3.0	450
BRS-II	8	Rough	#3—16 in. on centers	4.2	580
III-0.6-1.66	11	Rough	#3— 6 in. on centers	3.8	418

*Ratio of length of shear span to effective depth of tension reinforcement.

The AASHO procedure,[7] derived for steel-concrete beams, makes use of the useful capacity of mechanical connectors and of a variable factor of safety. The factor of safety is intended to furnish a connection capable of developing the ultimate flexural strength of the composite beam; it accounts for moving loads, for the different proportions of moments and shears resisted by the noncomposite and the composite sections, for the properties of the cross section and for the level of design stresses. The connection and the beam have to resist the same ultimate load. At working load, however, the beam is always resisting both the dead load and the live load while the connection may resist only the live load. In such case the factor of safety needed in the design of the connection is higher than that of the beam. If, however, the live and dead loads are distributed in the same manner along the beam, as is usually the case in building design, and if the assumption is made that all loads are resisted by the composite section, then the required factor of safety for shear connectors is equal to the factor of safety of the beam. The allowable values for various types of connectors may then be determined as the ratio of their strength to this constant factor of safety.

In the interest of simplicity, the committee recommends that the horizontal shear be computed from the elastic formula, using the vertical shear caused by *all* loads acting on the *beam* regardless whether shoring is used or omitted during construction(Section 204.2.1 and 403.2.1). It is important to note that the use of the *total* load in computing the horizontal shear is one of the basic assumptions in the derivation of the recommended design procedure.

Composite sections with steel beams designed as recommended by this committee usually have a factor of safety between 2.2 and 2.5. The useful strength of mechanical connectors, used on steel beams, may be determined from empirical formulas given in the AASHO specifications.[7] In using the AASHO formulas, the committee recognized their conservative nature in relation to static loading and, there-

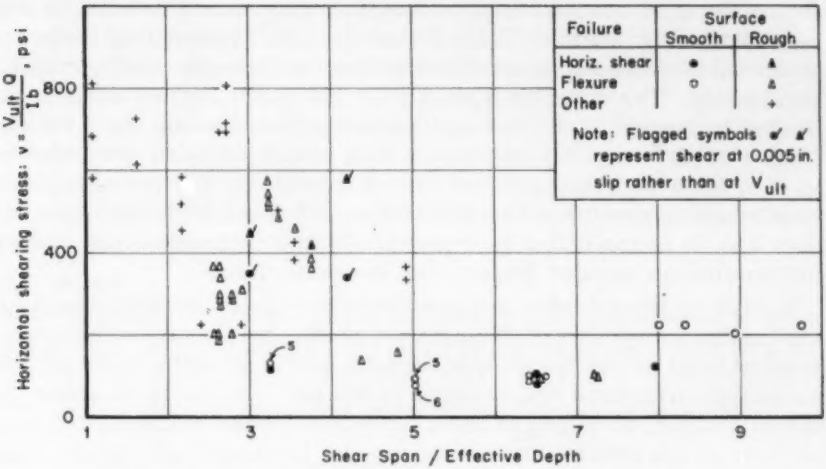

Fig. 3—Shear in composite concrete-concrete beams

fore, selected the safety factor of 2.0 as adequate when arriving at the recommended formulas for allowable loads (Section 404.1).*

The question of bond between the *steel* section and the *concrete* slab has always been a perplexing one. Tests have shown that it usually exists but no one can guarantee its duration. For this reason, the tentative recommendations give no credit to bond. Fully embedded beams constitute the only exception to this rule (Section 403.2.3).

For the transfer of shear between a *concrete* beam and a *concrete* slab the use of bond or of shear keys is recommended. Several experimental studies and experience have shown that bond in conjunction with steel ties provides a very effective and reliable connection. However, the tests have shown also that there is a practical limit to the capacity of the bond connection.

In the 78 tests of composite beams studied by the committee, nine beams failed in horizontal shear. The horizontal shear at which failure occurred (computed from the elastic formula), the type of surface, and a description of steel ties are given in Table 2 for all nine beams. The six specimens with a smooth surface failed at horizontal shearing stresses ranging from 78 to 350 psi, while the three specimens with a rough surface failed at horizontal shearing stresses between 418 and 580 psi. The remaining 69 specimens, which did not fail in horizontal shear, included both rough and smooth surfaces. The maximum horizontal shears obtained in the tests of all 78 beams are shown in Fig. 3.

The data for specimens with smooth contact surface cover well the range of moderate to large slendernesses (ratios of shear span a to ef-

*Preliminary data from an investigation now in progress at Lehigh University have indicated that ultimate strength of a composite beam might be developed with a substantially smaller number of connectors than that obtained by the procedure recommended at present.

fective depth of tension reinforcement d ranging from 3 to 9.75). It may be noted in Fig. 3 and in Table 2 that the data representing horizontal shear failures suggest a possible decrease of strength with increasing slenderness. The data for specimens with rough contact surface are limited to the range of small and moderate slendernesses (a/d ranging from 1 to 5); only two specimens with rough surfaces were slender ($a/d = 7.25$) and these did not fail. Furthermore, all three specimens with rough surface that failed in horizontal shear had a/d ratios less than 4.25. It is hoped that an investigation now in progress will provide information on slender beams with rough surfaces.

In view of the evidence discussed briefly in the preceding paragraph the committee considered it advisable at this time to base the recommended bond values for smooth surfaces on the strength of 80 psi, and for rough surfaces on the strength of 400 psi. The factor of safety for beams designed according to these recommendations ranges from 2.0 up. In view of the uncertainty concerning the bond strength in the range of larger a/d values, a factor of safety of 2.5 was used to obtain the allowable bond stresses (Section 205.3.1) for the rough surfaces. For smooth surfaces, better covered by the available test information, a factor of safety of 2.0 was selected.

The committee strongly recommends the use of steel ties crossing the contact area in all composite concrete T-beams. For light concrete joists and for slabs with precast beams completely embedded on three sides, this recommendation may be too severe. The committee considered such construction outside the scope of the tentative recommendations.

The minimum amount of ties recommended by the committee is based on the value suggested by Committee 323.[1] A small increase is suggested in bond values for beams with ties in excess of the minimum. This latter allowance is based on the results of recent tests of push-off specimens.[8]

In view of a lack of experimental data, the committee is not prepared to make any detailed recommendations concerning the design of shear keys.

Deflection

As a general rule, composite beams which are shallower and employed on longer spans than conventional beams are more deflection sensitive. The use of shores reduces the total deflection but creates construction problems. If the shore is properly placed, the deflection is a function of the load, span, and stiffness of the composite section. If the shore is driven too tight, the energy stored in the prefabricated beam will produce an additional effective load which will increase the deflection of the slab. Furthermore, if the floor is screeded level and the beam is shored high, the slab will be made too thin at midspan with resulting loss of strength.

In the case of an unshored beam, the deflection must be computed separately for the dead load—resisted by the prefabricated beam alone—and for the live load—resisted by the composite section. The total deflection is greater than for a beam built with shores but the floor may be screeded level at its final elevation. The beam itself must be checked for deflection under its weight plus that of the wet concrete and formwork to assure that excessive thickening of the slab does not occur at midspan. The tentative recommendations carry warnings about these potential problems of deflection.

Other factors

The question of deformational stresses was investigated in some detail. Such stresses are always present in composite members at working load stress levels. However, they are wholly internally balanced and hence have no effect upon the ultimate capacity. The committee felt that in ordinary buildings deformational stresses may be safely ignored except in unusual cases.

Creep of concrete affects the deflections of a composite beam. In cases such as beams carrying heavy masonry partitions, the long-time effect could cause serious cracking of the partitions even though the strength of the beam would be unaffected. The simplest manner of correcting the deflection computation for this long-time effect is to reduce the value of the modulus of elasticity of the concrete. It is recommended to use one-half the short time value of the concrete modulus to account for creep deflections.

The tentative recommendations contain a few guide lines for continuous design. The recommendations concerning the elastic properties of composite beams for the purpose of frame analysis follow the current practice for reinforced concrete T-beams. On the other hand, the recommendations for the design of the negative moment sections are in accord with the current bridge practice.

Finally, the recommendations concerning the effective slab width are based on the current practices both for composite beams and for reinforced concrete T-beams.

The recommendations have been prepared as a result of a need for a guide. Composite construction is being employed more and more as its advantages are being demonstrated.

The task of the committee is by no means finished. Its further work and direction of its investigations will be influenced by the reaction of the engineering profession to these tentative recommendations.

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This report was submitted to letter ballot of the committee which consists of 18 members; 17 members returned their ballots, of whom 16 have voted affirmatively and 1 negatively.

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Title No. 57-30

Concrete and Concrete Materials for Glen Canyon Dam

By WALTER H. PRICE, L. P. WITTE, and L. C. PORTER

Considerations involved in the preparation of specifications for aggregates, cement, and pozzolans for Glen Canyon Dam, which will be 710 ft high and contain about 5,000,000 cu yd of concrete, are discussed. Investigations which led to the selection of materials and the decision to remove soft, undesirable particles from the aggregates by heavy media processing are described. The aggregate processing plant, batching and mixing plant, and transportation and handling of the concrete are briefly described.

■ GLEN CANYON DAM, a concrete arch type being constructed on the Colorado River in Arizona, will be 710 ft high and 300 ft thick at maximum section, 1500 ft in length, and 35 ft thick at the crest. Approximately 5,000,000 cu yd of concrete will be required in its construction, and it will impound some 28,000,000 acre-ft of water in its 186 mile long reservoir which, when full, will cover about 150,000 acres. Merritt-Chapman and Scott Corp. was awarded a $108 million contract for construction of the dam and powerplant on Apr. 29, 1957. Twenty-five hundred days, nearly 7 years, are allowed for completing this contract. The Glen Canyon unit, including the dam, power plant, appurtenant works, and access roads will cost about $325 million.

This dam, 12 miles south of the Utah border and 135 miles north of Flagstaff, Ariz., the nearest railhead, is one of the key features of the Colorado River Storage Project, which was authorized by the 84th Congress, and will be used to implement some of the provisions of the Colorado River Compact. Essentially, the purpose of Glen Canyon Dam is twofold: (1) to provide storage of water during wet years to assure delivery at Lees Ferry for consumption by the lower Colorado River basin states of 75,000,000 acre-ft of water during any period of 10 consecutive years; plus, under certain circumstances, additional water for delivery to Mexico; and (2) through the generation of power, to provide funds to help finance participating irrigation projects located in the upper basin states.

Much has been accomplished at the damsite to date. The town of Page, with all facilities of a modern community, has come into being for housing government and contractor personnel. A steel arch vehicle

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L. P. Witte is assistant to chief research engineer, USBR, Denver. Active on ACI Committee 115, Research, and 212, Admixtures, he joined the USBR concrete laboratory as a junior engineer assigned to aggregate investigations in 1934. Except for a 2-year period, he has been with the Bureau continuously since that time. From the initial assignment to the present, his experience has included the testing and research of aggregates, pozzolans, and concrete for many Bureau concrete structures.

L. C. Porter, member of ACI Committee 621, Aggregates, is head of the aggregate and durability unit, Concrete Laboratory Branch, USBR, Denver. Joining the Bureau in 1946, he has since been engaged in materials and concrete testing and research in the engineering laboratories.

All three men are active ACI members.

bridge 1028 ft long has been completed across Glen Canyon. Two cableways of 50-ton and one of 25-ton capacity are in use. Diversion tunnels have been driven, lined with concrete, and the river has been diverted around the damsite. Excavation of the keyways and base is complete. A concrete aggregate processing plant, including heavy media processing features, has been installed. The batching and mixing plant has been completed, and concrete placing in the dam started.

Further afield, at Clarkdale, Ariz., a new cement mill has been erected to furnish some 3,000,000 bbl of cement. Also, a plant for processing natural pozzolanic materials in the amount of 220,000 short tons or 1,170,000 bbl is in operation north of Flagstaff.

Common and rock excavation of about 1,322,000 cu yd and 3,514,000 cu yd, respectively, was required before placement of concrete in dam was started June 17, 1960. Other significant items of construction materials include approximately 3000 tons of tubing and fittings for cooling and grouting concrete; 10,000 tons of penstock and outlet pipes; and 3000 tons of gates, hoists, and valves.

AGGREGATES

The infrequent occurrence of large aggregate deposits in northern Arizona and southern Utah offered little choice in the selection of an aggregate source. Wahweap deposit, the source of aggregate being used for construction, was initially tested as early as 1948 and was recognized at that time as a potential source of aggregate for the dam. After construction of the dam was authorized, sources within an area of 160 miles of the damsite were investigated, but none proved superior to the Wahweap deposit. Wahweap, extending from 6 to 13 miles northwest of the damsite, is a long, narrow deposit approximately 1000 to

1500 ft wide and about 7 miles long, and is the only deposit of natural aggregates within an economical distance of the damsite containing sufficient material for construction of the dam and appurtenant works.

Based on usual Bureau grading requirements for concrete aggregate, Wahweap pit run material contains insufficient 3- to 6-in. cobbles and a surplus of No. 4 to ¾-in. material (Table 1). The sand is deficient in the intermediate sizes, especially the No. 30 to No. 16 size fraction, and contains an excess of pan material and material retained on the No. 100 and No. 8 mesh sieves.

TABLE 1 — INDICATED PIT RUN GRADING IN AREA A WAHWEAP DEPOSIT* AND SPECIFICATIONS GRADING LIMITS

Screen size or number	Individual percent by weight retained on screen	
	Specifications limits	Indicated pit run grading
3 - 6 in.	13 to 30	10
1½ - 3 in.	20 to 35	21
¾ - 1½ in.	20 to 35	27
3/16 - ¾ in.	20 to 35	42
No. 4	0 to 5	..
No. 8	5 to 15†	16
No. 16	10 to 25†	10
No. 30	10 to 30	7
No. 50	15 to 35	17
No. 100	12 to 20	32
Pan	3 to 7	18

*Deposit indicated to contain 37 percent sand.

†If the individual percent retained on the No. 16 screen is 20 percent or less, the maximum limit for the individual percent retained on the No. 8 screen may be increased to 20 percent.

In addition, excessive amounts of lightweight material, which produce "popouts" on the surface of concrete when frozen and thawed, are contained in the aggregate, especially in the minus 1½-in. size fractions. Other deleterious materials contained in the deposit include appreciable quantities of soluble sulfates and rock types known to be reactive with alkalies in portland cement.

Ground water which underlies much of the Wahweap deposit at depths of from 8 to 12 ft contains up to 2200 ppm of sulfates, and the sand contains up to 1000 ppm, or 0.10 percent of sulfates. Washing the aggregates with clean water reduced the sulfate content of the aggregate well below 0.1 percent, the maximum amount which it was certain could be tolerated with safety. To assure aggregates of suitable quality from the sulfate standpoint, this figure (0.1 percent) was established in specifications as the maximum permissible average sulfate content of the sand at the batching plant. The specifications limitation is stated as follows: "Sand delivered to the batching plant may be rejected if it contains more than 0.15 percent soluble sulfate for any one sample or more than 0.10 percent for an average of at least 9 out of 10 consecutive test samples of finished sand, when samples are taken hourly." This limitation, of course, is to prevent extensive reuse of water for washing the aggregates which might build up harmful concentrations of sulfate.

As previously mentioned, the coarse aggregate in the deposit contains only 10 percent of 3- to 6-in. cobbles. Past experiences have shown that 25 to 30 percent of the coarse aggregate should be in the

3- to 6-in. size fraction to produce the most desirable and usually the most economical mass concrete. The higher percentages of cobbles reduce the amount of cement and mortar in the concrete with consequent benefits. A suitable supplemental source of cobbles, either natural or quarried, could not be located within an economical haul distance of the damsite. Additional cobbles in the amount desired could be obtained from the Wahweap deposit by scalping the cobbles from some 10,000,000 cu yd of additional material, but as will be seen later, this means of obtaining cobbles could not be justified on the basis of economy. The deposit contains a small quantity of rock larger than 8 in. in size, and use of this oversize rock, either in its natural state or as crushed aggregate, was considered but rejected because it is composed primarily of slabby, friable sandstone. Crushed coarse aggregate is permitted provided it meets all requirements of the specifications and is from a source other than the Wahweap deposit. Tests showed that the coarser sizes of sand and the finer sizes of gravel contain an objectionable amount of soft material. Because of this, it was decided that the deficiencies in the middle sizes of sand could not be supplied by crushing the coarse sands or gravel. There is an excess of sand in the deposit, and a better quality sand results if the coarser sizes are wasted. Furthermore, crushing contributes to the harshness of the concrete mix.

Mix investigations

Design criteria required that concrete for the dam should develop 3000 psi compressive strength in 180 days as determined from tests of sealed 18 x 36-in. test cylinders (Fig. 1) cured under a temperature cycle expected to occur in the dam. To satisfy this requirement, the compressive strength, based on a coefficient of variation of 15 percent, the average for Bureau projects, should average 3450 psi. These 18 x 36-in. test cylinders are commonly used in the Bureau laboratory to measure the indicated strength of mass concretes, as this size is the smallest which will give a representative value for concrete containing 6-in. maximum size aggregate. Such specimens, which are sealed against external effects of moisture and air and which are cured under a temperature cycle similar to that expected to occur in the dam, duplicate, as nearly as possible, the actual condition of concretes in the interior of the massive structure.

Further, considerations of the many unknown factors involved with extremely lean concretes dictated that in no event should the mass concrete for Glen Canyon Dam contain less than 2½ sacks of cementitious materials per cu yd of concrete, the cementing materials being portland cement or a combination of portland cement and pozzolan. Because of the low durability and higher permeability of this lean concrete, it was decided that the exposed faces of the dam contain not less than 3½ sacks of cementing materials (portland cement plus pozzolan).

Although the actual cement and pozzolan to be used in construction were not available when specifications were prepared, Type II cement and pozzolan, a pumicite having properties characteristic of materials available in the vicinity of the jobsite and likely to be used in the work, were available and used in preliminary mix investigations. Results of these investigations showed that minimum strength requirements could be met through the use of the 2 sacks of portland cement plus 1 sack of pozzolan per cu yd of concrete having a cobble content as low as 13 percent, the maximum amount of cobbles obtainable through efficient operation of the deposit. Because adequate strength could be obtained from low cobble concrete with this amount of portland cement, the increased cost of processing for additional cobbles was not justifiable. Laboratory tests indicate that a compressive strength of 3570 psi at 180 days' age will be developed by Wahweap aggregate concrete containing 13 percent cobbles, 2 sacks of portland cement, and 1 sack of pumicite-type pozzolan per cu yd.

Fig. 1—Instrumentation for determining modulus of elasticity on 18 x 36-in. concrete cylinders. This size specimen is also used for compressive strength determinations

Mixes comparing concretes made with finely graded, washed pit run sand and sands graded to usual Bureau specifications requirements indicated appreciably greater economy for concrete containing the graded sand; also, it was desirable to waste excess sizes of sand through vigorous washing to reduce the amount of soft lightweight material. Consequently, usual Bureau grading requirements were specified for the sand.

During the comprehensive tests of aggregate for Glen Canyon Dam, the aggregate evaluation freezing and thawing test in concrete was performed. In this test, it was noticed that after a few cycles of freezing and thawing, the surface of concrete cylinders containing the Wahweap aggregates became severely pock-marked and scaled, the surface deterioration being identified with individual pieces of aggregate ranging in size from ¾-in. gravel, the largest size contained in the specimen, to approximately No. 30 size sand (Fig. 2). As the test continued, this scaling increased over the entire surface of the test specimens in a relatively few cycles. However, after this initial deterioration of the surface, additional freezing and thawing damage to the concrete progressed slowly, and the over-all resistance of Glen Canyon concrete was judged to be very good.

Fig. 2—Disfigured surface of concrete subjected to 50 cycles of freezing and thawing indicates need for beneficiating Wahweap aggregate

These results indicated that freezing and thawing would not affect the structural performance of Glen Canyon air-entrained concrete; however, weathering might seriously affect the appearance of the dam and exposed concrete in other features. Because of the importance of this structure, every effort was made to minimize the disfiguring pop-outs and scaling, as well as to improve the over-all quality of the concrete.

Beneficiation of aggregate

Offending rock types responsible for the surface deterioration during freezing and thawing were identified petrographically as calcareous siltstones, calcareous cherts, shaly limestones, and ferruginous concretions. These rock types, with the exception of ferruginous concretions, are appreciably lighter in weight than are the sound constituents. Surface deterioration from freezing and thawing of concrete containing aggregate from which lightweight material had been removed was greatly reduced (Fig. 3). Consequently, specifications require that the No. 8 to 1½-in. size fractions to be used in all exposed concrete for the job should be processed to remove material having a specific gravity of 2.50 or less. Laboratory tests indicate that about 20 percent of the

Fig. 3—Concrete with Wahweap aggregate after 250 cycles of freezing and thawing exposure. Top row — Contains heavy media beneficiation aggregates. Bottom row — Contains untreated aggregate

materials in the No. 8 to 1½-in. size range are lightweight, the coarse fractions containing somewhat less and the fine fractions somewhat more than this percentage.

Beneficiation, by heavy media or other commercially proven and effective methods, was limited by specifications to those sizes since there was virtually no lightweight material in the plus 1½-in. gravel and no known effective and economical process that could remove the lightweight material from the minus No. 8 sand sizes. It is further specified that the No. 8 sand which has been so beneficiated be recombined with the other sizes of sand not so processed. In addition, specifications stipulate that any size fraction of aggregate which requires beneficiation may contain not more than 2.0 percent lightweight material having a specific gravity of 2.50 or less.*

To accomplish removal of lightweight material, the contractor has elected to employ a heavy media flotation process.† This relatively recent application to the beneficiation of concrete aggregates is an adaptation of a mining process which has been used for a number of years. Briefly, the process involves introducing clean aggregate into a heavy liquid suspension composed of water and finely ground ferrosilicon and magnetite where the lightweight particles are floated off and wasted while the heavy, more sound particles sink and are reclaimed for use in concrete. Heavy media processing at the above-mentioned specific gravity will not remove the unsound ferruginous

*Higginson, E. C., and Wallace, G. B., "Control Testing for Separation of Lightweight Material from Aggregate," *ASTM Bulletin*, No. 243, Jan. 1960, Appendix, p. 67.

†"Heavy Media Treatment of Gravel," *Circular* No. 55, National Sand and Gravel Association, Mar. 1953.

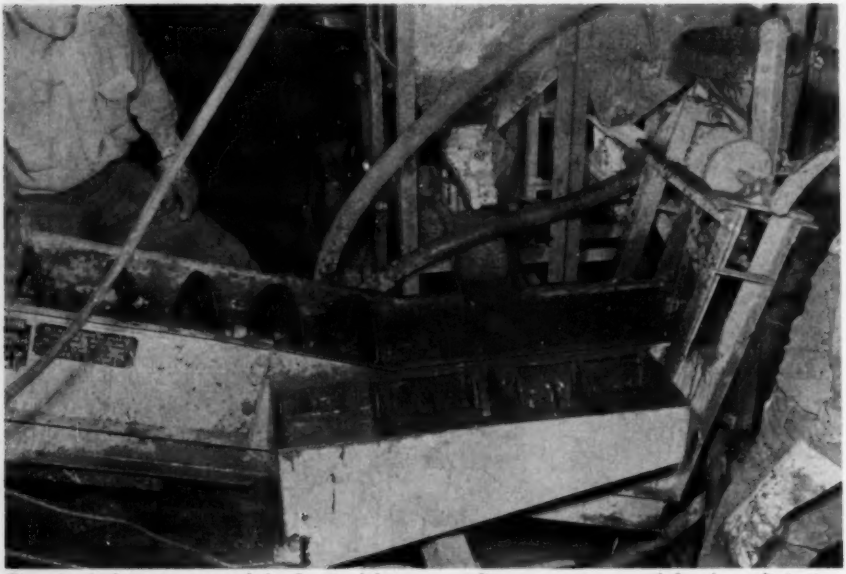

Fig. 4—Laboratory model of spiral heavy media separator used for beneficiating aggregates. Lightweight particles are floated off over the weirs shown in the foreground and are wasted. Heavy particles sink to the bottom of the heavy media pool, are reclaimed by the spiral, are rewashed to remove media

concretions as these are about the same weight as the heavy portion of the aggregate; these constitute but a minor percentage (0.4 percent) of the total, and their presence is not viewed with too much alarm.

The contractor is processing aggregates in his main plant in the lower end of Wahweap Creek about 6 miles northwest of the damsite. This plant, which is eight times as large as the pilot plant which furnished aggregates for preliminary concreting operations, has an input capacity of 1400 tons per hr and a usable output of 800 tons per hr. Two 6½-cu yd draglines excavate the aggregate which is transported by 36-in. conveyor belt to the plant. The pit run material is passed through a grizzly to remove the small amount of oversize, which is wasted, then is conveyed by belt to the screening, washing, and as required, heavy media plants, for processing. Water for washing is obtained from four wells, drilled in the vicinity of Wahweap Creek, having a combined capacity of 6000 gpm. To supplement this meager supply, which must provide mixing water also, a small amount of water from Wahweap Creek is blended with water from the wells. An earth storage reservoir of 4,500,000 gal. capacity was constructed to stabilize the amount of water available. After washing and sizing, the aggregates are conveyed to bunkers of 3000 to 4000 ton capacity located astride a drive-under roadway. Loading of the 30 ton capacity trucks is accomplished by the driver through one or more of the 15 ft long gates in the bunkers. The time

required to load is approximately 10 sec, and a fleet of 12 of the 30-ton trucks is expected to handle all demands for concrete aggregate.

Heavy media separation is being accomplished using two units of different type (Fig. 4 and 5). The 3/16- to 1½-in. material is heavy media processed through a spiral separator and the sink portion of the aggregate rewashed and rescreened. The No. 8 to 3/16-in. fraction of the sand is heavy media separated through a cone separator. Effective clean separation of the No. 8 to 3/16-in. fraction of sand for heavy media processing requires special precautions in screening by the contractor. Following heavy media processing, this sand fraction is reblended with the finer fraction not heavy media processed. Reblending is accomplished by metering appropriate amounts of heavy media processed No. 4 to No. 8 and nonheavy media processed minus No. 8 sand onto

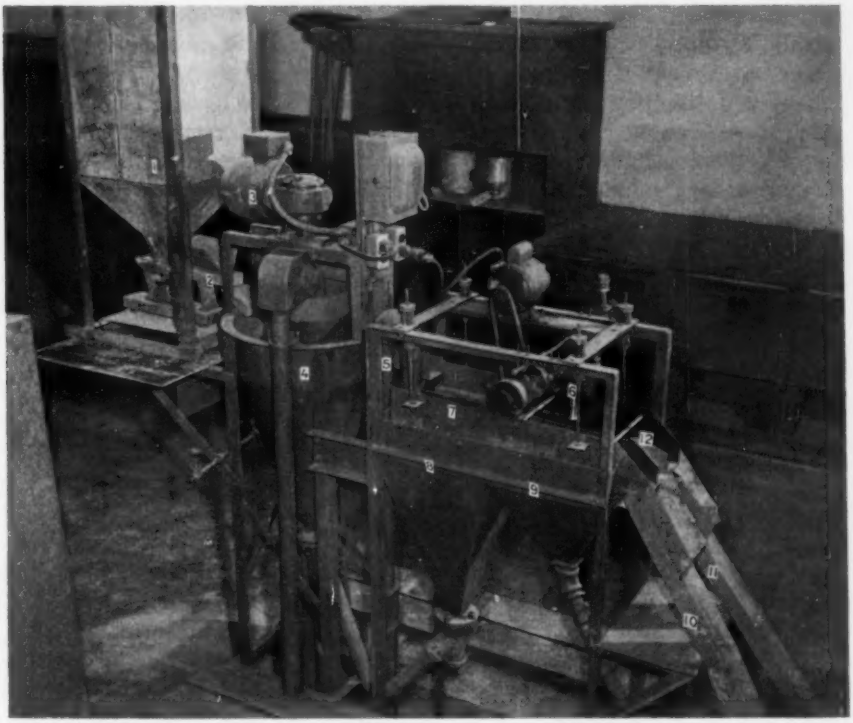

Fig. 5—Laboratory model of cone heavy media separation plant used to beneficiate concrete aggregates: (1) feed hopper, (2) adjustable vibrating feeder, (3) gear head motor (prime mover for a pair of sweeps which continually scrape the sidewalls of the separatory cone to prevent the settling of medium), (4) separatory heavy media cone, (5) airlift for sink material, (6) wash-water spray nozzle, (7) vibrating screen, (8) media drain sump, (9) wash sump, (10) sink product spout, (11) float product spout, (12) partition to separate sink and float products

a common conveyor belt running through the reclaiming tunnel beneath the two stockpiles of sand.

Cementing materials

It is Bureau policy to have the contractor furnish most of the materials to be used in the construction, including cement and pozzolan. However, for the Glen Canyon project, far removed from any established sources of portland cement or pozzolan, it was believed to be advantageous for the government to furnish the cement and pozzolan on contracts separate from the general construction contract.

Pozzolan

There are a number of advantages to the use of pozzolan in a massive structure such as Glen Canyon Dam. It improves the workability of the fresh concrete, reduces heat of hydration as compared to the amount of heat generated by portland cement required to produce equal strengths at later ages, reduces permeability, and usually costs less than the cement it replaces. The resistance to freezing-thawing of concrete containing pozzolan and cured for the short periods specified on the job is low as compared to that of straight portland cement concrete, and for that reason, pozzolan has not been specified for the concrete in the power plant and other comparatively thin and exposed appurtenant structures.

From the beginning, it was anticipated that a pozzolan would be used at Glen Canyon Dam. Known commercial sources of pozzolan located several hundred miles distant and natural deposits in the general vicinity of the damsite were investigated.

Altogether, some 97 samples from 74 sources (nine from commercial sources) including volcanic materials, shales and clays, diatomite and diatomaceous earth, and fly ash were tested. Many of the natural sources lie within a radius of 200 miles from the damsite. Of these, the shales and clays, which in addition to grinding require calcining to activate, were nearest to the damsite, but most costly to process. Next in order of processing cost and proximity to the damsite were the volcanic materials which generally require grinding only. Fly ash, which would require no processing, was located extremely far from the project and was consequently burdened with high transportation costs.

Specifications for pozzolans are contained in the appendix. Specifications were written around "natural pozzolan" and "fly ash;" and with but three exceptions, the requirements for the two types of material were the same. The exceptions are tabulated below:

	Natural pozzolan	Fly ash
Loss on ignition, not more than	10 percent	5 percent
Blaine specific surface, sq c per g, not less than	7000*	3000
Water requirement, not more than	115 percent	103 percent

*Limit applicable to volcanic ashes, pumice, and pumicites only.

In this specification, the usually required 75 percent reduction in expansive reaction was lowered to 60 percent because of the protection provided by the low-alkali cement specified. In addition, the usual requirement on amount of air-entraining agent was removed entirely.

Cement

Approximately 3,000,000 bbl of portland cement at an estimated peak rate of production approaching 120,000 bbl per month will be required to build the Glen Canyon Dam. To obtain information concerning the availability of cements in the general area, a letter seeking answers to specific questions was addressed to 14 cement companies in the West and Southwest. Half of the letters of reply indicated interest, of varying degrees, in furnishing the cement for the project. Also, the consensus was that no premium would be charged for Type II over Type I cement, but that a premium of from $0.10 to $0.30 per bbl *would* be charged if low alkali was specified. With this information at hand, cement specifications were drafted.

Initially, Type I cement was considered because, at least until recently, it had been obtainable at lower cost than Type II. Numerous analyses and tests of Type I cements from mills in the area were performed to determine heat of hydration, both by the method of heat of solution and by adiabatic temperature rise. Results indicated that the temperature rise of Glen Canyon concrete using a typical Type I cement would be above that desired with the 7½-ft lifts permitted in the construction of the dam, unless a cement was employed which approached a Type II cement in C_3A and C_3S contents. Consequently, Type II cement was specified. Because of potential reactivity of the Wahweap aggregate, the Type II cement specified was also required by specifications to be of low alkali content; that is, 0.60 percent or less equivalent soda and to conform to Federal Specification SS-C-192b, and applicable paragraphs covering alkali content and heat of hydration at 7 days.

Because of the long haul from the nearest existing cement plants and because of the large quantity of cement required, it was indicated that some savings might be realized by constructing a plant nearer the damsite. Specifications for the cement were issued approximately 18 months prior to scheduled need of cement in the dam concrete, and the successful bidder elected to build a new plant at Clarkdale, Ariz., some 160 miles south of the damsite.

All acceptance tests other than tests for false set and temperature are being made on samples taken during filling of cement bins at the cement mill and reserved for government use. Acceptance tests for false set and temperature of bulk cement are made at the time of delivery to the damsite but before the cement is unloaded. The maximum temperature at which cement will be accepted by the government and

delivered to the contractor is 170 F. This temperature limitation, it is believed, will reduce the possibility of the cement developing false set in the contractor's bins. Also, it provided the contractor a value for designing the refrigeration facilities required to obtain a temperature of 50 F in the concrete as placed.

Concrete

The contractor is required to erect and maintain a complete concrete plant, including equipment for processing, conveying, and stockpiling aggregates; storage and handling facilities for cement, pozzolan, and air-entraining agent; mixing and batching facilities; refrigeration equipment for cooling aggregates and water; and a cableway or other means of conveying and placing the concrete.

Concrete placing is scheduled at the rate of approximately 9000 cu yd per day. To achieve this goal, the contractor is using a mix plant having six 4-cu yd tilting mixers. The plant has a 3000-ton aggregate storage bin divided into eight 375-ton compartments to accommodate both heavy media processed and untreated sands and all size fractions of coarse aggregate. Bins are charged by a single conveyor belt from the permanent stockpile area, alternating between sand and coarse aggregate as necessary. Both the cement and the pozzolan active storage silos at the batch plant are of 700 bbl (2800 cu ft) capacity. Mixers discharge into three wet batch hoppers totaling 65 cu yd capacity. These, in turn, discharge into 12 cu yd capacity railroad transfer cars. Concrete is delivered to either of two 50 ton capacity cableways and transferred into 12-cu yd buckets for depositing in the dam.

The cableways for the dam are situated one above the other; both having traveling head and tail towers, and either line will be able to service all areas of the dam and powerplant. Working together, the two cableways will be able to handle a 100-ton load. The upper, main cableway stretches 2050 ft between towers on opposite sides of the canyon. This main line, the largest ever built, is 4 in. in diameter, weighs 38 lb per ft, has a strength of 880 tons, and will operate at a tension of 320 tons. Hoist speeds of up to 700 ft per min are possible and believed to be the fastest of any cableway presently in operation. A smaller 25 ton capacity cableway is being used, principally for transporting concrete in 8-cu yd buckets for placement in the powerplant structure.

Concrete in Glen Canyon Dam will be both precooled and postcooled by means and for reasons discussed hereafter. Precooling to assure placing temperatures of 50 F or less will be accomplished by sprinkling and submerging of the coarse aggregate with chilled water enroute to the batching bins, where it will be further cooled by refrigerated air blasts, by cooling the mixing water, and by the addition of chip ice to the mix. Postcooling, to control temperature rise resulting from hydration of the cement, will be accomplished by means of cool water

circulating through a system of pipes embedded in the concrete blocks. Later, these pipes will be used to cool the concrete to open contraction joints so that they can be grouted.

Precooling of materials with the resulting relatively low temperature concrete is necessary to keep the maximum temperature of the concrete (placing temperature plus temperature rise after placing) to about 75 F or less and thus minimize the danger of the blocks, some of which are 7½ x 210 x 70 ft in size, cracking when they are cooled to as low as 40 F before grouting the contraction joints.

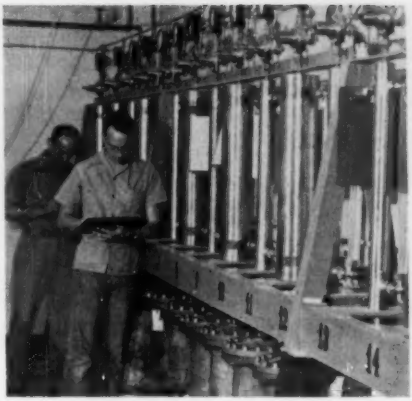

Fig. 6—Apparatus used to determine water permeability of concrete

Curing of concrete, in general, will be accomplished through the use of water. Exceptions to this are (1) exterior wall surfaces of powerplant, machine shop, etc., that will be exposed to view, (2) spillway tunnels or other work not exposed to public view, and (3) galleries and shafts, if capable of sealing, to provide 90 percent relative humidity.

Item (1) above must be cured with gray pigmented membrance curing compound; Item (2) will accomplish curing with either water or membrane curing compound; and Item (3) requires no curing other than that stipulated.

Cleanup of horizontal construction joints prior to placing concrete follows the usual established pattern. Wet sandblasting, thorough washing with air-water jets, and removal of joint cleanup water and debris shall be performed immediately prior to concrete placing.

Comprehensive mix program

The cement for Glen Canyon Dam is being furnished from a new mill at Clarkdale, Ariz., and the pozzolan from a pumicite deposit located near Flagstaff, Ariz. When all job materials were available, samples were obtained for the comprehensive concrete mix program now being conducted. The investigations will determine the most desirable mix proportions for both interior and exterior Glen Canyon Dam concretes, based on workability, strength development, elastic and thermal properties, creep, permeability, drying and autogenous shrinkage, and durability. A face concrete will be employed at Glen Canyon Dam, and this mix will be 1 sack of portland cement per cu yd richer than mass concrete for the interior mix. A variety of test specimens ranging up to 18 x 36-in. cylinders is utilized, and some tests will continue even after the dam is completed to study the behavior of Glen

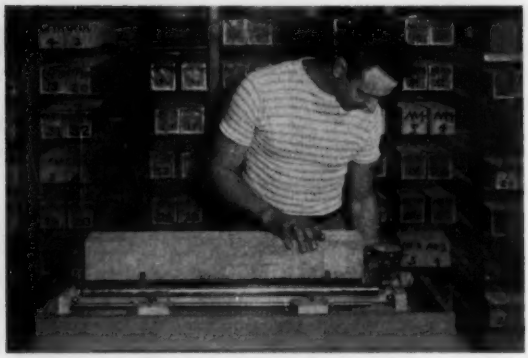

Fig. 7 — Length measurements are used to determine drying shrinkage of concrete

Canyon concrete over an extended period of time (Fig. 6, 7, and 8).

Based on results obtained from preliminary concrete mixes, including mixes with different cement contents and aggregate gradations both with and without pozzolan additions, the general limits in Table 2 were established and included in specifications to aid in preparation of bids.

Although its use was not required by specifications, a water-reducing, retarding agent was used in the concrete, containing 2½-in. maximum size aggregate, for the tunnel lining at Glen Canyon Dam. In this case, the contractor requested and was given permission to use a lignin type agent which was compatible with the project cement being used at that time.

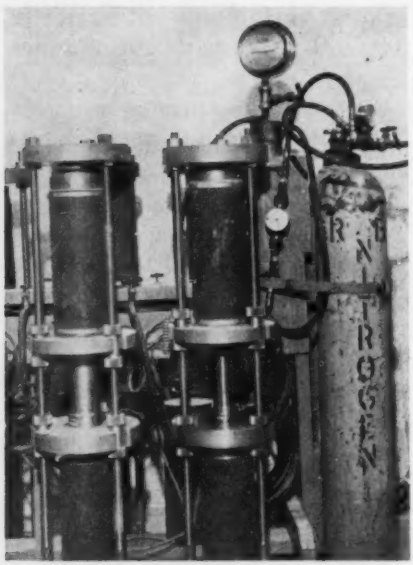

Fig. 8—Apparatus used to determine creep characteristics of concrete subjected to sustained loads

Approximately 0.25 lb of agent per sack of cement was used and resulted in a 9.1 percent reduction in cement and water requirements and concomitant net savings of $116,000, but a reduction at 28 days' age of 410 psi compressive strength to an average of 4340 psi. No apprehension was felt over this reduction because the average strength exceeded the established average design strength of 3450 by about 900 psi. Had a lesser savings in cement been realized, higher strengths would have resulted until at zero reduction in cement, a 20 percent increase in strength over that developed by the control concrete would have been expected. With these advantages, together with the possibility of re-

TABLE 2—GENERAL LIMITS

Nominal maximum size aggregate (concrete)	Cementing materials, portland cement plus pozzolan (approximate)		Sand, percent of total aggregate	Coarse aggregate, percent coarse aggregate only			
	Total sacks* per cu yd. of concrete	Percent pozzolan (by weight of cement)		3/16 to ¾ in.	¾ to 1½ in.	1½ to 3 in.	3 to 6 in.
6 (interior)	2½ to 3½	20-35	20-28	20-35	20-35	20-35	13-30†
6 (exterior)	3½ to 4½	20-35	20-28	20-35	20-35	20-35	13-30†
3	3½ to 4½	0-25	26-31	25-40	20-40	20-40	
1½	4 to 6	0-25	31-37	45-60	40-55		
¾	5 to 7	0-25	40-46	100			

*Based on 94 lb per sack of cementing materials.
†The aggregate grading data indicate that there is a shortage of the 6-in. coarse aggregate in the Wahweap aggregate deposit and the percentage of this size required in concrete will be held to the minimum shown.

ducing the occurrence of "cold" joints through retardation of the setting time of the concrete, the use of a water-reducing, retarding agent in the mass concrete of the dam proper is potentially attractive and is being considered.

SUMMARY

This recounts the highlights of some of the many factors which were considered in the specific field of concrete and concreting materials for Glen Canyon Dam. It is by no means a complete accounting as many details purposefuily have been omitted. However, it will serve, we think, to point out methods used in solving some of the many and varied problems encountered in the design and construction of a project of this magnitude. It must be remembered that similar problems and similar details had to be worked out for the many other facets of the job, such as foundation investigations, grouting requirements, design of the dam and power plant, the spillway, the outlet works, and other features. Only by careful coordination to provide answers to these many problems and design considerations can a project such as Glen Canyon Dam be successfully built.

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APPENDIX — USBR POZZOLAN SPECIFICATIONS

C-1. Definition—The term pozzolan is used to designate a siliceous or siliceous and aluminous material, which in itself possesses little or no cementitious value but will, when combined with portland cement, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties.

The detail requirements hereinafter specified are for natural pozzolan (pozzolan other than fly ash) and for fly ash. Natural pozzolans include such materials as some diatomaceous earths; opaline cherts and shales, tuffs; and volcanic ashes or pumicites, any of which is pozzolanic in nature and may or may not require calcination and/or grinding in order to meet the requirements of these specifications. Fly ash is the fine ash precipitated from the flue gas at the stacks of

certain types of coal-burning furnaces and shall meet the requirements of these specifications. Pozzolans such as clays and shales, will require calcination and grinding to meet the detail requirements of these specifications; other pozzolans, except fly ash, may require calcination and grinding. In general, for any pozzolan requiring calcination, it is necessary for the pozzolan to be calcined at a temperature of not more than 1800 F and not less than 1400 F in order for the pozzolan to meet the detail requirements of these specifications.

C-2. Detail requirements—Only one type of pozzolan having the same general chemical and physical characteristics shall be furnished under this invitation. The blending of two or more types of pozzolanic material to form a single pozzolan to be supplied under these specifications will not be permitted. The temperature of pozzolan when delivered to the Government shall be not more than 170 F.

Pozzolan, when tested in accordance with the methods of test hereinafter specified, shall meet the following requirements for chemical composition, physical properties, and reactivity:

Chemical composition	Natural pozzolan	Fly ash
Silicon dioxide (SiO_2) plus aluminum oxide (Al_2O_3) plus ferric oxide (Fe_2O_3) not less than, percent	75.0	75.0
Magnesium oxide (MgO), not more than, percent	5.0	5.0
Sulfur trioxide (SO_3), not more than, percent	4.0	4.0
Loss on ignition, not more than, percent	10.0	5.0
Moisture content, not more than, percent	3.0	3.0
Exchangeable alkalies as Na_2O, not more than, percent	2.0	2.0

Physical properties	Natural pozzolan	Fly ash
Fineness:		
Specific surface, sq c per g (air permeability fineness method of test), not less than	7000*	3000
Material retained on No. 325 mesh sieve, percent, not more than	15	15
Compressive strength:		
With portland cement, percent of control, 28 days, not less than	85	85
With lime, 7 days, minimum psi	900	900
Change of drying shrinkage of mortar bars, percent shrinkage of pozzolan bar minus percent shrinkage of control bar, not more than	0.04	0.04
Water requirement, not more than, percent	115	103

*Limit applicable to volcanic ashes, pumice, and pumicites only.

The specific gravity of individual samples from any one source shall not vary more than 3 percent from the average established by the ten preceding samples or by all preceding samples if the number is less than ten.

Reactivity	Natural pozzolan	Fly ash
Reduction of expansive reaction at 14 days, not less than	60 percent	60 percent

Any pozzolan having abnormalities with respect to hardening, setting, rate of heat of hydration, or any other abnormality, which may or may not be

specifically covered in this invitation, as determined by preliminary test, will not be approved.

Pozzolan, when tested in accordance with the provisions of Paragraph C-5 of this invitation, shall be considered as meeting the detail requirements if the following criteria are met:

(a) The average of all test results obtained for each specified test shall meet the specified limit.

(b) The average of any five consecutive tests for each specified test shall meet the specified limit.

(c) Not more than one test in ten consecutive tests for each specified shall have a value of less than 90 percent of the specified value when the specified value is on the basis of "not less than" or "minimum," or a value of more than 110 percent of the specified value when the specified value is on the basis of "not more than."

C-3. Information for bidders — Bureau of Reclamation tests have been made on samples of pozzolan taken from numerous undeveloped deposits within a radius of approximately 275 miles from the damsite. From results of tests performed, the most promising undeveloped deposits are as follows:

Undeveloped deposit area in the vicinity of	Indicated type of material	Usual processing required
Cedar Ridge, Ariz., approximately 40 to 70 miles south of damsite, along U. S. Highway No. 89	Bentonite clay	Calcination and grinding*
Flagstaff, Ariz.	Pumice rock	Grinding*
Beaver, Utah	Pumice	Grinding*
Milford, Utah	Volcanic ash	Grinding*
Steamboat, Ariz.	Rhyolitic volcanic ash	Grinding*
Panaca, Nev.	Rhyolitic volcanic ash	Grinding*

*Including drying as necessary to meet the specified moisture content limitation.

Calcareous shale, such as contained in an undeveloped deposit located approximately 12 miles northwest of the damsite, or any other calcareous shale, will not be approved as a suitable material to be supplied under this invitation.

Results of Bureau of Reclamation tests of pozzolan for Glen Canyon Dam are available for examination by bidders at the office of the Assistant Commissioner and Chief Engineer, Denver Federal Center, Denver 2, Colo., or at the office of the Project Construction Engineer, Page, Ariz. The Government does not guarantee that the information and data pertaining to these tests are representative of the conditions that may be encountered in the field and the successful bidder shall be entitled to no additional compensation if further development of any of the deposits or if more detailed information and data obtained by the contractor reveals that the Government's preliminary analysis is not representative of general conditions in the deposit.

C-4. Preliminary tests and approval — The successful bidder shall, within 30 days after date of award of contract, submit for preliminary tests and approval a representative 100-lb sample of pozzolan from each source from which he proposes to supply pozzolan under this invitation: *Provided,* That the maximum number of sources which will be tested by the Government, free of charge to the successful bidder, will be three: *Provided further,* That a charge not to exceed $500 will be made against the successful bidder for testing each pozzolan source in excess of three. If, at any time after the 30-day period, the successful bidder decides to obtain pozzolan from a source other than the source or sources originally proposed, a representative sample of pozzolan from this source shall

be submitted for preliminary tests and approval at the earliest practicable date and in any event, not less than 8 months before the first shipments of pozzolan will be required. Approval of samples furnished shall not relieve the contractor of the responsibility of furnishing pozzolan in compliance with the requirements of this invitation. The successful bidder will be charged an amount not to exceed $500 for the cost of testing for any sample submitted after the 30-day period, regardless of whether use of materials from another source was necessitated by reason or rejection of materials from the source or sources originally proposed, or for any other reason.

Any royalties or other charges required to be paid for materials taken from deposits not owned by the Government available to the Bureau of Reclamation shall be paid by the contractor. Approval of a pozzolan deposit shall not be construed as constituting approval of all materials taken from the deposit, and the successful bidder will be held for the specified quality of all materials furnished. If the pozzolan deposit to be used in supplying pozzolan under this invitation is on lands owned by the Government, and available to the Bureau of Reclamation, no charge will be made to the contractor for materials taken from the deposit and shipped in accordance with this invitation.

C-5. *Inspection* — The pozzolan will be sampled and tested* by the Government. A 15-lb sample will be taken representing approximately each 150 tons. Individual tests will be made on each sample to determine the following properties: specific surface (as required), material retained on No. 325 mesh sieve, compressive strength with lime, water requirement, specific gravity, loss on ignition, and moisture content.

Tests on all other properties, except for temperature, will be made on composite samples. Tests for temperature of pozzolan will be made on a sufficient number of truckloads or carloads, as determined by the contracting officer, to assure that the pozzolan will meet the requirements of the specifications. Composite samples will be prepared by thoroughly mixing equal amounts from each of eight consecutive samples in sufficient quantity to make the required determination. Where practicable, as determined by the contracting officer, acceptance tests will be made on samples taken at the production source at the time final processing of the pozzolan has been completed but before shipment. If pozzolan is loaded into trucks or cars on completion of final processing or where no processing is required, or where pozzolan is required prior to completion of tests, it may, subject to approval of the contracting officer, be shipped on the supplier's certification that it conforms in all respects to these specifications. Permission to ship pozzolan either before completion of tests or without testing shall in no way relieve the contractor of the responsibility for furnishing pozzolan meeting the requirements of these specifications.

*A photostatic copy of the USBR "Methods of Test for Pozzolan" may be obtained from ACI headquarters at $1.

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Title No. 57-31

Effect of Draped Reinforcement on Behavior of Prestressed Concrete Beams

By JAMES G. MacGREGOR, METE A. SOZEN, and CHESTER P. SIESS

Tests on 19 simply supported pretensioned concrete beams with draped prestressed reinforcement are described and compared with the results of tests of similar beams with straight prestressed reinforcement. The principal variables included: concrete strength, steel percentage, length of shear span, and the angle and type of drape profile. Web reinforcement was used in only five beams.

In general, it was concluded that draping the longitudinal wires did not increase either the inclined cracking load or the shear strength of the prestressed concrete beams tested. Instead, the trend of the test results indicated a reduction in both the inclined cracking load and the ultimate strength of the beams with draped wires. A comparison of the behavior of beams with draped and straight wires showed that the detrimental effect of the drape on "shear" strength could be ascribed to the earlier formation of flexural cracks in regions of combined bending and shear and the consequent earlier development of inclined cracks. For extreme combinations of the critical variables, an inclined crack occurred prior to the formation of flexural cracks in its vicinity, in which case draping the wires caused an increase in strength which could be estimated on the basis of an uncracked section analysis.

The beams with draped reinforcement required more web reinforcement to produce a flexural failure than similar beams with straight reinforcement.

■ TESTS OF PRESTRESSED CONCRETE BEAMS conducted in the last few years have shown repeatedly that reinforcement should be provided in prestressed members to resist inclined tensile stresses as well as horizontal tensile stresses. Two different types of reinforcement are commonly used for this purpose: stirrups and draped reinforcement. The term "draped reinforcement" refers to longitudinal prestressed reinforcement which is curved in elevation so that its centroid is closer to the top of the beam at the supports than it is at midspan. Stirrups of various grades of steel and of various shapes are used solely to carry "shear." On the other hand, the longitudinal reinforcement may be draped primarily to relieve tensile stresses in the top flange or to alleviate anchorage stresses. The slope of the tendons results in an upward component of prestressing force which is customarily subtracted from the total downward shear acting on a section.

A trio of ACI members report on research . . . **James G. MacGregor**, assistant professor of civil engineering, University of Alberta, Edmonton, Alta., Canada, was previously research associate in civil engineering, University of Illinois . . . **Mete A. Sozen** is an associate professor of civil engineering, University of Illinois, Urbana. His work involves research in prestressed and reinforced concrete; he is active on ACI Committee 335, Deflection of Concrete Building Structures . . . **Chester P. Siess**, professor of civil engineering, University of Illinois, is well known to ACI members. He has been active on numerous ACI technical committees, including chairmanship of several, covering such subjects as codes, prestressed concrete, slabs, bond, and fatigue.

The tests described in this paper were carried out to determine whether linearly draped reinforcement improves the strength and behavior of prestressed concrete beams. Twenty-two tests of simply-supported pretensioned beams are reported. The principal variable in the test series was the profile of the prestressing steel. Twelve reinforcement profiles, each consisting entirely of straight line segments, were used.

Notation

Designation of test specimens — Each beam is designated by a group of letters and two pairs of numerals; for example, BD.14.26. The letters and numerals have the following significance:

First letter (BD.14.26)
- A—rectangular beam
- B—I-beam, 3-in. web
- C—I-beam, 1¾-in. web

Second letter (BD.14.26)
- D—draped reinforcement
- V—draped reinforcement and web reinforcement
- W—web reinforcement

(The second letter is excluded if the beam has neither draped reinforcement nor web reinforcement.)

First numeral (BD.14.26)
- 1—prestress greater than 90,000 psi
- 2—prestress less than 90,000 psi

Second numeral (BD.14.26)
- 3—27-in. shear span, equals 3/12 span length
- 4—36-in. shear span, equals 4/12 span length

The second pair of numerals represents the value of $Q = E_s p/f_c'$ to two significant figures, where E_s and f_c' are in psi.

Symbols — The following symbols have been used in this paper:

a = shear span, the distance between the support and the nearest load point
A_c = gross area of cross section
A_s = total area of prestressed reinforcement
A_v = area of one stirrup
b = top flange width
b' = web thickness
d = maximum effective depth of the reinforcement in the beam
E_s = modulus of elasticity of steel
f_c' = compressive strength of concrete determined from 6x12-in. control cylinders
f_r = modulus of rupture of concrete determined from 6 x 6 x 24-in. control beams loaded at the third points of an 18-in. span
f_{se} = effective prestress
F_{se} = effective prestressing force
f_t = tensile strength of concrete
f_{vy} = yield point stress of web reinforcement
h = over-all height of beam
M = bending moment
M_c = applied bending moment at inclined cracking

M_c' = computed bending moment at inclined cracking for a beam with straight longitudinal reinforcement

M_u = ultimate moment measured in test

M_{uf} = computed ultimate moment for flexural failure

M_{us} = computed ultimate moment for shear failure

$p = A_s/bd$ = reinforcement ratio

$Q = E_s p/f_c'$

$r = A_v/bs$ = web reinforcement ratio

s = spacing of stirrups

V = shear force

V_c = applied shear at inclined cracking

V_c' = computed shear at inclined cracking for a beam with straight longitudinal reinforcement

V_d = upward component of prestressing force

V_f = shear required to cause a flexural crack in the shear span at a distance x from the load point

V_n = net shear or effective shear at inclined cracking

V_w = computed shear at web-shear cracking

V_u = ultimate shear measured in test

$x = h/4 + a/6$ = distance between the initiating crack and the load point

ϕ = drape angle = angle between the resultant prestressing force and the horizontal

SPECIMENS, INSTRUMENTATION, AND TEST PROCEDURE

Description of beams

The I-beams and rectangular beams tested were nominally 6 x 12 in. in over-all cross section and 10 ft 1½ in. or 10 ft 8 in. in over-all length. The nominal cross-sectional dimensions are shown in Fig. 1. Beam properties are listed in Table 1. All the beams were pretensioned, the wires being draped

TABLE 1 — PROPERTIES OF BEAMS

Mark	Concrete strength* f_c', psi	Flange width b, in.	Web thickness b', in.	Effective depth d, in.	Steel area A_s, sq in.	A_s/bd, percent	Effective prestress f_{se}, ksi	Drape angle† ϕ, deg	Web reinforcement rf_{vy}, psi	Wire lot
AD.14.37	3260	6.00	6.00	10.15	0.242	0.398	106	6.45	—	11
B.14.34	2640	6.05	3.10	10.30	0.181	0.290	115	0	—	12
B.14.41	2890	6.05	3.00	10.00	0.242	0.399	114	0	—	12
BD.14.18	6280	5.95	2.86	10.11	0.237	0.386	123	2.70	—	13
BD.14.19	6280	5.95	2.90	10.20	0.242	0.398	112	5.00	—	12
BD.14.23	3870	6.00	3.00	10.10	0.181	0.300	99	9.13	—	11
BD.14.26	3460	6.00	3.00	10.10	0.181	0.298	116	9.95	—	11
BD.14.27	3400	6.00	3.00	10.10	0.181	0.298	111	2.22	—	11
BD.14.28	3320	6.00	3.00	10.10	0.181	0.300	118	1.53	—	11
BD.14.34	2700	6.05	3.00	10.22	0.181	0.293	110	1.68	—	11
BD.14.35	2610	6.05	2.95	10.10	0.181	0.297	108	6.28	—	11
BD.14.42	2870	6.00	2.90	10.10	0.242	0.400	107	2.38	—	11
BD.24.32	3800	6.05	3.00	10.10	0.242	0.395	81	6.45	—	11
BV.14.30	4020	5.95	2.95	10.10	0.242	0.403	123	3.25	48.6	11
BV.14.32	3800	5.90	2.85	10.13	0.242	0.403	112	3.23	54.5	12
BV.14.34	3620	5.95	3.00	10.15	0.242	0.400	124	2.70	54.0	12
BV.14.35	3410	5.95	2.92	10.20	0.242	0.398	115	5.36	69.5	12
BV.14.42	2910	6.00	2.88	10.15	0.237	0.387	120	6.80	60.5	13
C.13.23	3730	6.05	1.79	10.38	0.181	0.288	119	0	—	12
CD.13.24	3670	5.90	1.77	10.56	0.181	0.290	124	3.40	—	12
CD.13.25	3460	6.05	1.82	10.46	0.181	0.286	118	2.85	—	12
CD.14.34	2560	6.00	1.75	10.22	0.181	0.296	105	1.88	—	11

*The strength of the concrete in the compression zone.
†The drape angle, ϕ, is the angle between the centroid of reinforcement and the horizontal.

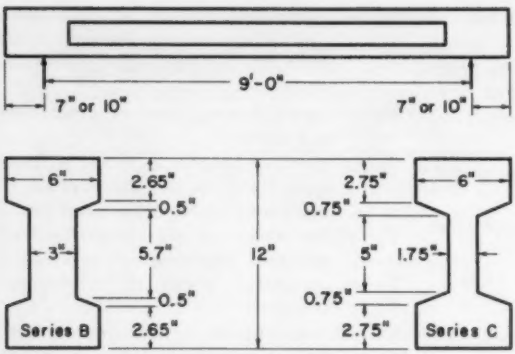

Fig. 1—Nominal dimensions of I-beams

from the load points in every case. Details of drape profiles are shown in Fig. 2 and are described in Table 2.

Materials

Concrete was made with Type III portland cement, a Wabash River sand having a fineness modulus of 3.0 to 3.3, and a ⅜-in. maximum size Wabash River gravel. Proportions by weight of the concrete mixes ranged from 1:4.2:4.2 for low strength concretes to 1:2.3:2.7 for high strength concretes. The corre-

TABLE 2 — PROPERTIES OF DRAPE PROFILES

Drape profile*	Mark	Number of wires	Number of draped wires	Height of center of gravity of steel			Angle of draped wires, deg	Drape angle ϕ, deg
				At support		At midspan		
				Draped wires, in.	All wires, in.	All wires, in.		
A	BD.14.28	6	6	2.96	2.96	2.0	1.53	1.53
B	BD.14.27	6	6	3.40	3.40	2.0	2.22	2.22
	BD.14.42	8	8	3.50	3.50	2.0	2.38	2.38
C	CD.13.24	6	6	3.23	3.23	1.63	3.40	3.40
D	BD.14.19	8	8	5.14	5.14	2.0	5.00	5.00
	BV.14.35	8	8	5.36	5.36	2.0	5.36	5.36
E (Midheight)	AD.14.37	8	8	6.07	6.07	2.0	6.45	6.45
	BD.24.32	8	8	6.07	6.07	2.0	6.45	6.45
	BD.14.35	6	6	5.96	5.96	2.0	6.28	6.28
	BV.14.42	8	8	6.30	6.30	2.0	6.80	6.80
F	BD.14.23	6	6	7.8	7.8	2.0	9.13	9.13
G	BD.14.26	6	6	8.33	8.33	2.0	9.95	9.95
W	BD.14.34	6	4	4.14	3.04	1.87	3.42	1.88
	CD.14.24	6	4	4.14	3.04	1.87	3.42	1.88
X	BD.14.18	8	4	5.37	3.69	2.0	5.36	2.70
	BV.14.34	8	4	5.37	3.69	2.0	5.36	2.70
Y	CD.13.25	6	4	3.81	3.12	1.67	4.27	2.85
Z	BV.14.30	8	4	6.07	4.04	2.0	6.45	3.25
	BV.14.32	8	4	6.07	4.04	2.0	6.45	3.25
S (Straight)	B.14.34	6	—	1.87	1.87	—	—	—
	B.14.41	8	—	2.00	2.00	—	—	—
	C.14.34	6	—	1.64	1.64	—	—	—

*Profiles A through G have only draped wires. Profiles W through Z comprise both straight and draped wires. Profile S designates straight reinforcement only. Profiles C and Y have 27-in. shear spans.

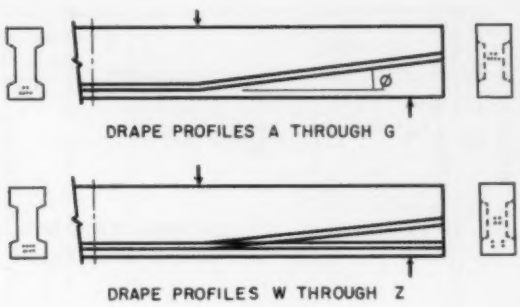

Fig. 2—Typical drape profiles

sponding water-cement ratios by weight ranged from 1.00 to 0.59. Slumps ranged from 1 to 6 in.

Cold drawn and stress relieved high tensile strength single wire meeting the requirements of ASTM A 421-59T was used for longitudinal reinforcement. The wires used were received in three shipments designated as Lots 11, 12, and 13. Stress-strain curves for the three lots are shown in Fig. 3. The modulus of elasticity of all the wires was about 30,000,000 psi. To improve bond characteristics, all wires were allowed to rust slightly. Loose rust was removed with a wire brush.

Stirrups were made of No. 8 black annealed wire. The stress-strain curve for this wire is shown in Fig. 4. The stirrup wire was also rusted.

Fabrication and curing

The longitudinal wires were prestressed in a movable frame made of two 11 ft 6 in. lengths of 3-in. pipe and two transverse bearing plates. This frame fitted around the form for the beams. The reinforcement was tensioned one wire at a time and anchored by hardened chisel-steel nuts on threads cut on each end of the wire. The stress in each wire was determined by measuring compressive strain in calibrated aluminum dynamometers slipped over each wire at the end opposite to that at which the wire was tensioned.

The reinforcing wires were tensioned in their uppermost position and then pulled down to their final position by a welded steel saddle at each load point. The saddles fitted over the wires and passed through holes in the bottom of the form where they were held in position by nuts bearing on the underside of the form. The form was mounted on a stiffening beam which prevented it from

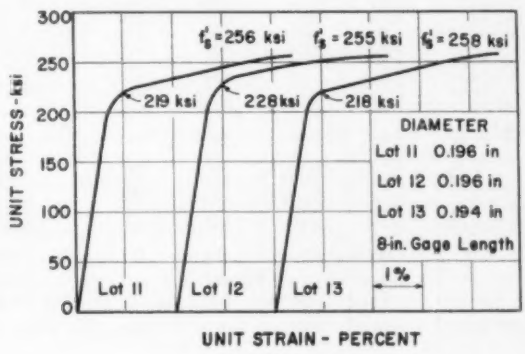

Fig. 3 — Stress-strain relationships for prestressing wires

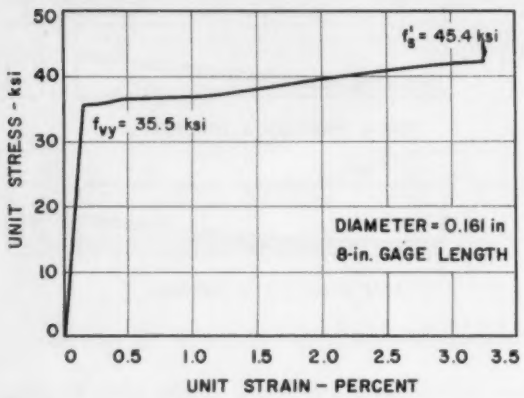

Fig. 4 — Stress-strain relationship for stirrup wire

deflecting excessively under the draping forces. The initial prestress in the wires was chosen so that the additional increment added by draping brought the total prestress up to the desired level.

The concrete was mixed for 3 to 6 min in a 6-cu ft nontilting mixer. Two batches were required for each beam. The first batch was placed in the bottom two-thirds of the form, the second batch in the top third. High frequency vibrators were used to consolidate the concrete in all cases. The beams were cast in steel forms. The forms were stripped after 2 days and the beams were covered with wet burlap for about 3 days and then allowed to air-dry until tested. The control beams and cylinders were cured in the same way.

The strength properties of the concrete in each batch are summarized in Table 3. Each compressive strength listed is the average from at least four

TABLE 3 — PROPERTIES OF CONCRETE MIXES

Mark	Compressive strength f_c', psi		Modulus of rupture f_r, psi		Cement:sand:gravel by weight	Water/cement by weight		Age at test, days
	Batch 1	Batch 2	Batch 1	Batch 2	Batches 1 and 2	1	2	
AD.14.37	2700	3260	300	282	1:4.21:4.58	0.91	0.91	12
B.14.34	3090	2640	340	275	1:4.07:4.42	0.79	0.79	8
B.14.41	3000	2890	358	358	1:4.07:4.42	0.79	0.79	7
BD.14.18	6390	6280	517	438	1:2.90:3.20	0.72	0.71	6
BD.14.19	6720	6280	519	519	1:2.80:3.00	0.72	0.74	7
BD.14.23	4210	3870	337	310	1:4.05:4.30	0.78	0.78	15
BD.14.26	3160	3460	383	392	1:4.24:4.60	0.84	0.84	8
BD.14.27	3850	3400	442	416	1:4.27:4.60	0.79	0.79	9
BD.14.28	4230	3320	457	367	1:4.01:4.36	0.77	0.78	8
BD.14.34	2720	2700	404	350	1:4.25:4.56	0.79	0.79	8
BD.14.35	2610	2610	375	400	1:4.17:4.55	0.92	0.92	8
BD.14.42	2980	2870	491	491	1:4.20:4.51	1.00	1.00	8
BD.24.32	3090	3800	416	375	1:4.27:4.60	0.81	0.81	8
BV.14.30	4200	4020	346	350	1:3.93:4.17	0.82	0.82	20
BV.14.32	4210	3800	420	451	1:3.90:4.20	0.84	0.83	6
BV.14.34	3800	3620	500	416	1:4.00:4.20	0.86	0.85	5
BV.14.35	3340	3410	508	425	1:3.90:4.20	0.83	0.83	5
BV.14.42	3090	2910	455	450	1:3.90:4.21	0.90	0.90	6
C.13.23	3460	3730	495	425	1:4.40:4.45	0.87	0.83	13
CD.13.24	3850	3670	467	437	1:4.40:4.45	0.90	0.90	14
CD.13.25	3020	3460	408	417	1:4.45:4.45	0.85	0.85	11
CD.14.34	2660	2560	417	420	1:3.83:4.25	0.91	0.94	6

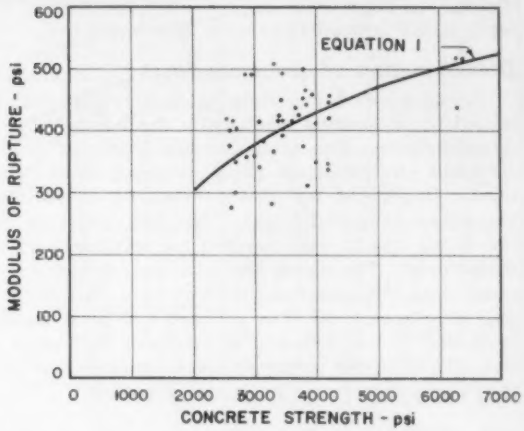

Fig. 5 — Variation of modulus of rupture with concrete strength

tests on 6 x 12-in. cylinders. Each modulus of rupture listed is the result of one 6 x 6-in. beam loaded at the third points of an 18-in. span. In Fig. 5, the measured moduli of rupture are plotted against concrete strength and are compared to the expression

$$f_r = \frac{3000}{4 + \frac{12{,}000}{f_c'}} \quad \text{(1)}$$

presented in Reference 1 for the modulus of rupture of concrete with ⅜-in. maximum aggregate size, where f_r and f_c' are in psi.

Instrumentation

Strains in the concrete at the top of the beam were measured with Type A3 SR-4 electrical resistance strain gages. Strains in the tension steel were measured with Type A7 SR-4 gages. Deflections were measured at midspan and at the load points with 0.001-in. dial indicators. In some of the beams, 0.001-in. dial indicators were mounted on the wires at the end of the beam to detect end slip.

Test procedure

The beams were tested 1 to 3 days after releasing the prestress in order to keep the prestressing losses at a minimum. Loads were applied by a hydraulic ram reacting against a testing frame and were measured by an elastic-ring dynamometer having a sensitivity of 110 lb per dial division. All but three of the beams were tested with equal loads applied at the third points of a 9-ft span. The remaining three beams, C.13.23, CD.13.24, and CD.13.25, were tested on an 8 ft 6 in. span with a single concentrated load placed 27 in. from the reaction. In these tests, the load was applied first near one end until inclined cracking occurred, after which the loading jack was moved near the other end of the beam which was then loaded to failure. External stirrups were used to restrain the cracks which had formed under the first loading. The results for these tests are listed as CD.13.24a, CD.13.24b, etc., for the two ends.

In all tests, failure was reached in five to ten increments of load. After each load increment, deflections and strains were measured and the cracks were marked. In addition, midspan deflections were measured at frequent intervals

during the application of each load increment. The concrete control specimens were tested immediately after the beam test.

Determination of prestress level

During prestressing, draping, and release, the force in each wire was determined by measuring electrically the compressive strain in calibrated aluminum dynamometers. The instantaneous losses at release were measured with Type A7 SR-4 electric strain gages mounted on the wires at midspan. The computed elastic losses and the losses measured at the time of release by the gages on the wires compared closely. The uniformity of the prestress along the span of the drape beams was checked by additional SR-4 gages mounted outside each drape point. The losses due to creep, shrinkage, and relaxation were estimated using data obtained from control tests on concrete cylinders, prestressed beams, and wire specimens. The estimates of total losses were corroborated by measurements of the increase in midspan deflection between times of release and test. The effective prestress levels listed in Table 1 were obtained by subtracting the total losses from the initial prestress.

BEHAVIOR OF TEST BEAMS

Modes of failure

The beams failed in either flexure or shear, the majority failing in shear.

A flexural failure is one in which bending stresses predominate. Such a failure will occur at a section of maximum moment, either by fracture of the steel or by crushing of the concrete in the compression zone over a flexural crack. All the test beams had sufficient longitudinal reinforcement so that a flexural failure would be expected to occur by crushing of the concrete over a crack with steel strains in the inelastic range. The distribution of strains over the depth of the beam at the section of failure remains nearly linear throughout the life of a beam failing in flexure.

A beam is said to have failed in shear if its failure is initiated by an inclined tension crack resulting from a combination of bending and shear stresses. This crack disrupts beam action and leads to a nonlinear distribution of strains over the depth of the section. The load and deflection at failure are both smaller than those for a flexural failure.

Development of cracks

Since a beam is primarily a flexural member, its dimensions are chosen on the assumption that ultimate failure will be in flexure. However, if an inclined tension crack forms before the flexural capacity is reached, and if the opening of this crack is not restrained by web reinforcement, the beam will be weakened. It is, therefore, important to know at what load and how an inclined crack forms.

When a prestressed concrete beam with a moderate amount of reinforcement is loaded to failure in flexure, its behavior goes through two or three different stages. Before cracking, the beam behaves "elasti-

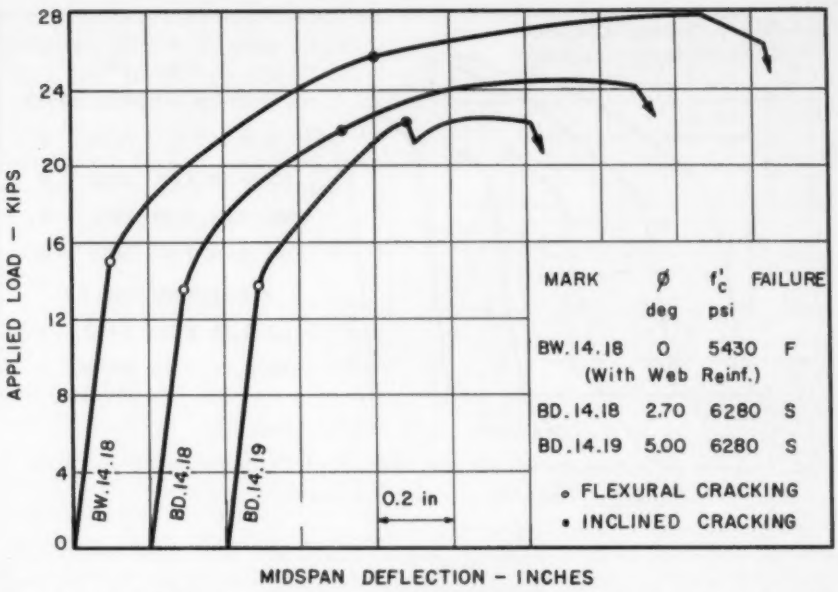

Fig. 6—Load-deflection curves showing the effect of drape angle on ductility

cally," the deflections being proportional to the applied loads. The first flexural cracks form vertically near the point of maximum moment and are followed by similar cracks throughout the constant moment region, if the beam has one. With further load, flexural cracks form in the shear spans, the parts of the beam between the loads and the supports. These cracks extend vertically to the level of the reinforcement and then tend to bend over toward the load points. During the flexural cracking stage, the rate of increase in deflection with respect to load increases continually as the load is increased. This phenomenon is due to a number of factors such as a shift in the neutral axis caused by cracking of the concrete and inelastic action in the concrete.

The third stage in the behavior of a beam failing in flexure is initiated by inelastic straining of the tension reinforcement and is marked by a rapid increase in deflections with respect to load. The load-deflection curve for BW.14.18 (Reference 2) in Fig. 6 illustrates the stages in the load-deflection behavior of a prestressed beam with enough stirrups to ensure a flexural failure.

If inclined cracks occur in a beam without web reinforcement, its strength and ductility are reduced. This may be seen from the load-deflection curves in Fig. 6 and 7. A beam failing in flexure may never develop inclined cracks and in some extreme cases beams failing in shear may not develop flexural cracks. In the beams tested, inclined

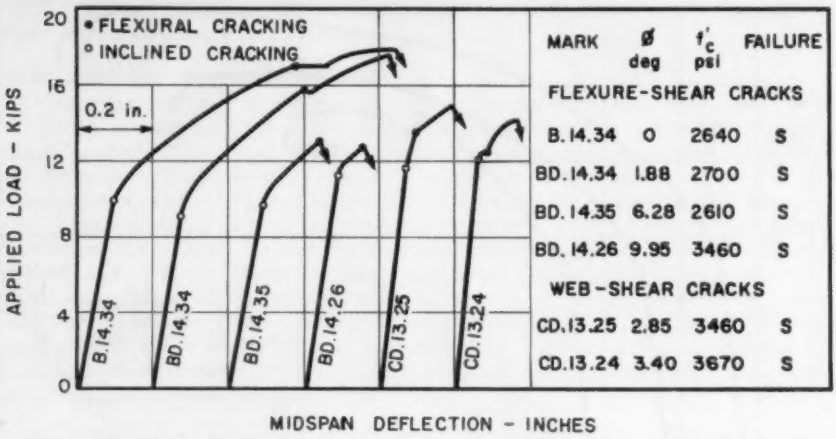

Fig. 7—Load-deflection curves showing the effect of drape angle and type of inclined cracking on ductility

cracks occurred in two different ways, which are discussed below. The strength and behavior were different for the two types of inclined cracking.

In beams with high prestress, thin webs, and short shear spans, the principal tensile stresses in the web may exceed the tensile strength of the concrete before flexural cracks occur in the shear span. An inclined crack which occurs in the web before flexural cracks appear in its vicinity is referred to as a "web-shear crack." This type of inclined crack is shown in Fig. 8a.

In beams with moderate prestress, relatively thick webs, and relatively long shear spans, flexural cracks will occur in the shear span before the principal tensile stresses in the web are high enough to cause web-shear cracks. If an inclined crack does occur in such a beam, it is either the extension of a flexural crack in the shear span or it occurs over or beside such a flexural crack. This type of inclined crack is referred to as a "flexure-shear crack," since the flexural cracks in the shear span contribute to the development of such a crack. The critical flexural crack which influences the formation of the inclined crack, is referred to as an "initiating crack." This type of inclined cracking is shown in Fig. 8b.

One of the beams with draped cables failed in flexure, three developed web-shear cracks, and 17 developed initiating cracks followed by flexure-shear cracks. In four of the latter, the initiating crack triggered the flexure-shear crack immediately (see BD.14.35 or BD.14.23 in Fig. 9), while in the remaining 13 beams, the load at inclined cracking was as much as 25 percent greater than that causing the initiating crack. In some beams, for example in BD.14.27 (Fig. 9), the inclined crack

was merely an extension of the initiating crack. However, in most cases the initiating crack developed, crossed the tension steel, and bent over toward the load point. The action of the initiating crack eventually raised the stresses in the web to a critical value which caused the flexure-shear crack to form.

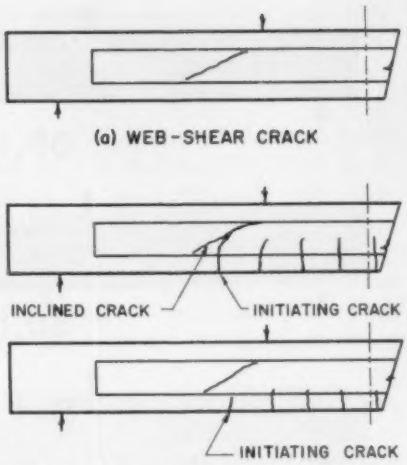

Fig. 8—Types of inclined cracks

In the beams which developed web-shear cracks and in many of the beams with flexure-shear cracks, there was no question about the magnitude of the inclined cracking load since the crack opened suddenly to its full height. In some cases, however, the flexure-shear crack developed gradually from an initiating flexural crack in the shear span. For such beams, the inclined cracking load was defined as the load, after the inclined crack started to develop, at which a break was observed in the plot of the steel strain versus the concrete strains measured over the top of the crack. This break indicated that the inclined crack was beginning to affect the distribution of strains over the depth of the section and thus the over-all behavior of the beam.

Behavior of test beams with draped wires

Behavior of beams with web-shear cracks—Three beams with draped wires, CD.13.24, CD.13.25 and CD.14.34, and one beam with straight wires, C.13.23, developed web-shear cracks before an initiating flexural crack could form. In these four beams, the first inclined cracks occurred close to the load at an angle of approximately 30 deg with the horizontal. The very brittle behavior of a beam developing web-shear cracks is shown by the load-deflection curve of Beam CD.13.24 on Fig. 7. The crack pattern in this beam at failure is shown in Fig. 9.

The first crack in CD.14.34 was a web-shear crack which originated near the bottom of the web, spread rapidly across the web toward the load point, and also caused splitting along the steel toward the support (Fig. 9). The bottom of this crack coincided approximately with the point where the draped prestressing wires entered the web. The failure was extremely brittle, occurring before flexural cracks had formed anywhere in the beam.

Behavior of beams with flexure-shear cracks—The behavior of the beams which developed flexure-shear cracks varied with the drape

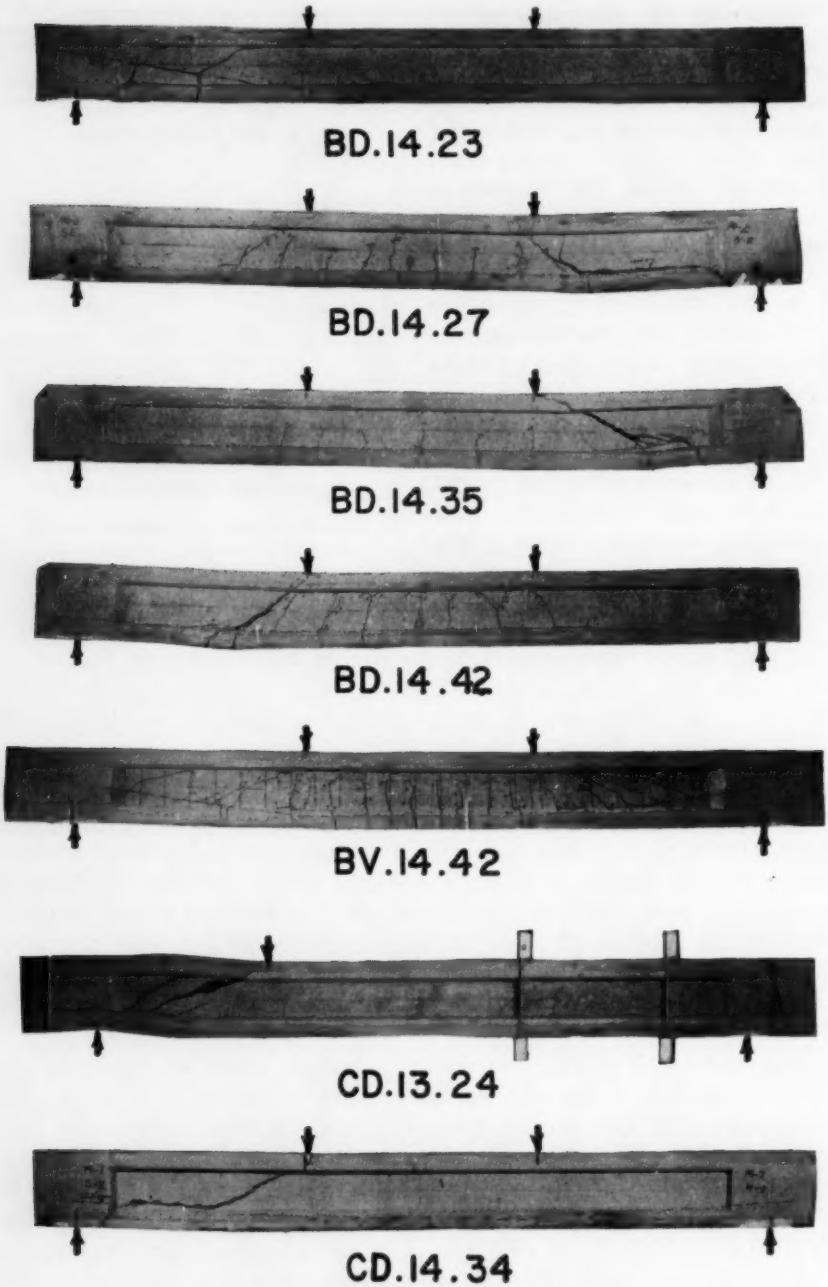

Fig. 9—Test beams after failure

profile; the beams with straight wires and the beams with the very high drapes represented the extremes in behavior. The principal difference in behavior for the different drapes was the rate at which the inclined tension crack developed.

All the beams with draped wires which developed flexure-shear cracks were loaded at the third points of a 9-ft span. The first flexural cracks occurred in the middle third of the span and, with further load, flexural cracks developed in the shear spans. The position of the flexural cracks in the shear spans differed from beam to beam. In beams with high drapes, the initiating crack occurred farther from the load point than in beams with low drapes. In each case, the load causing the initiating crack was lower than would be required in a similar beam with straight wires.

The behavior of the beams after formation of the initiating cracks was a function of the drape profile. For beams with straight wires and the beams with low drape profiles there was about a 25 percent increase in load between the formation of the initiating crack and the development of the flexure-shear crack. In these beams, the inclined crack crossed the reinforcing steel outside the load point at a distance approximately equal to the over-all depth of the beam (Beams BD.14.27 and BD.14.42 in Fig. 9).

For the beams with high drape angles, there was no increase in load between the formation of the initiating crack and the development of the inclined crack. In these beams, the cracks crossed the steel outside the load point at a distance approximately equal to 1½ times the over-all depth of the beam (Beam BD.14.23 in Fig. 9).

The longitudinal reinforcement ratio did not appear to affect the manner of crack development, nor did having some wires draped and others straight.

The effect of the drape angle on the ductility of the beams with all wires draped is illustrated by the load-deflection diagrams in Fig. 6 and 7. The curve for Beam BW.14.18 has been included in Fig. 6 for comparison. This beam had straight wires and enough stirrups to ensure a flexural failure. It should be pointed out that the first break in each curve does not reflect any effects of draping because it corresponds to first flexural cracking in the constant moment region where the reinforcement was parallel to the longitudinal axis of the beam. For beams having the same shear spans, the effect of increasing the drape angle is indicated by the reduction in both the inclined tension cracking load and the ductility.

Splitting along the reinforcement had a major effect on the behavior of beams with draped reinforcement. The splitting was especially serious in beams which had draped reinforcement passing through the web. The rate of splitting depended on the angle of drape, the

beams with high drapes splitting faster than the beams with low drapes.

Since the initiating and inclined cracks formed in the region where the prestressing wires were in the web in many of the I-beams, there was some question as to whether the rapid development of splitting near these cracks was a consequence of the reduced thickness of the web. For this reason, a rectangular beam, AD.14.37, was tested. This beam had eight wires draped to midheight over the support. When loaded at the third points of a 9-ft span, one end failed in shear very suddenly, leaving the remainder of the beam virtually undamaged. The failure load was about 60 percent of the load carried by similar beams having straight wires. After removing the beam from the testing frame, the cracked portion was removed and the remainder of the beam was restested on a 64-in. span with a single load at midspan. These two tests on this beam are referred to as AD.14.37a and AD.14.37b, respectively. In Beam AD.14.37b, one shear span had draped wires and the other had straight wires, and the first cracks were flexural cracks under the load point. The shear span in which the wires were draped developed a flexural crack before the other span, as would be expected. This crack initially rose as high as the steel. With further loading, the crack progressed toward the load, splitting occurred, and a flexure-shear crack developed in the draped span. Failure occurred by crushing of the compression zone over this crack. At failure there were no inclined cracks in the shear span with straight wires, showing that the inclined cracking load was lower in the shear span with draped wires than it was in the shear span with straight wires. The development and consequences of inclined cracks in this rectangular beam were thus similar to those observed in the I-beams, even though there was no section of reduced thickness in the rectangular beam.

Observed modes of failure

Beams with web-shear cracks—The actual mechanism of failure in shear depended on the type of inclined cracks which had developed in the beam. A web-shear crack causes a redistribution of the stresses in the shear span and, since there is no shear flow between the tension and compression flanges, beam action ceases and the beam carries load as a tied arch. The compression thrust line over the inclined crack is generally close to the crack and almost parallel to it. Hence, the thrust acts with a considerable eccentricity on the portion of the beam over the crack near the reactions. This was evidenced by tension cracks in the top flange. These cracks were followed immediately by crushing of the web as illustrated by Beam CD.13.24 in Fig. 9. Such a failure was sudden and destructive.

Beams with flexure-shear cracks—Beam action is destroyed in the shear spans of a beam after the formation of flexure-shear cracks.

After such a crack develops, the linear relationship between concrete and steel strains ceases to exist since the concrete strains are concentrated above the end of the inclined crack while the steel strains are distributed over a length at least equal to the horizontal projection of the crack. Thus, a small increase in steel strain over this length is accompanied by a proportionately larger increase in concrete strain in the compression zone over the crack. When the concrete strains reach a limiting value, crushing occurs, destroying the beam. This type of failure is known as a "shear-compression failure" and is discussed more fully in Reference 1.

If the prestressing steel becomes unbonded in the neighborhood of the inclined crack, the incompatibility of strains is magnified even more since the length over which the steel strains are distributed is increased by the additional unbonded length. If the tension reinforcement is "unbonded" by separation of the tension flange from the rest of the beam, this may constitute the primary failure before a shear-compression failure occurs. In about half of the beams tested, cracks occurred along the steel in the vicinity of the initiating crack, both during the interval between the formation of the initiating crack and the flexure-shear crack and after the flexure-shear crack had formed. In these beams, the steel in one-third to one-half of the shear span was unbonded by cracks along the steel when failure occurred.

Only five of the beams without web reinforcement failed in shear-compression. These beams were AD.14.37b, BD.14.18, BD.14.19, and BD.14.42 with draped wires, and B.14.34 with straight wires. The increases in load beyond inclined cracking were 1, 1, 5, 1, and 14 percent, respectively, for the beams listed. Studies made in Reference 1 showed that an I-beam with similar properties and no web reinforcement but with straight tension steel, should carry about 15 percent more load after inclined cracking before a shear-compression failure occurs.

In a number of beams, the inclined crack and the splitting along the steel shortened the anchorage zone so much that a secondary failure occurred in the wire anchorages. It should be emphasized that this occurred after the inclined cracks had formed and that the inclined cracking load was not affected by the failure of the anchorage bond.

In general, the beams with draped wires and web reinforcement failed in flexure. Exceptions to this were Beams BV.14.30 and BV.14.32. In both of these beams a long relatively flat inclined crack extending from the load point to the reaction occurred immediately before failure. Bond slip was measured in BV.14.32 *after* this crack occurred. It is not known whether BV.14.30 was also a bond failure, although this seems probable from the appearance of the failure. Beams BV.14.34, BV.14.35, BV.14.42, and the second test on BV.14.32 (BV.14.32b) all failed in flexure (see BV.14.42 in Fig. 9).

DISCUSSION OF TEST RESULTS

In this section the inclined tension cracking loads of prestressed concrete beams with straight wires and concentrated loads are discussed first. This is followed by a discussion of the similarities between the phenomena of inclined cracking in beams with straight wires and beams with draped wires, and the effect of draping the wires on the inclined cracking load. These two sections will deal with inclined cracking since the inclined cracking load represents the dependable service limit of a beam without web reinforcement. Furthermore, the tests reported by Hernandez[2] indicate that the amount of web reinforcement required in a prestressed concrete beam is critically influenced by the quanitative relation of the inclined cracking load to the flexural capacity.

Inclined cracking loads for beams with straight wires

Web-shear cracks—A web-shear crack is defined in this paper as an inclined crack which develops in the web of a beam before flexural cracks develop in its vicinity. Therefore, in the case of web-shear cracking, and only in this case, the principal tensile stresses computed for an uncracked section will approximate the stresses in the web of the beam at the time of inclined cracking. If it is assumed that web-shear cracking is a stress phenomenon and that the critical tensile strength of the concrete in the web can be determined or estimated, then it should be possible to compute the web-shear cracking load.

The constantly changing combinations of shearing stress and flexural stress, and near the loads and reactions, bearing stress, make it difficult to determine the location and magnitude of the maximum principal tensile stress in a beam at the time of inclined cracking. Thus, the inclined crack may originate at, above, or below the elastic centroid, and anywhere along the length of the shear span where the stresses are large enough. Since the web-shear cracks observed in the tests presumably formed where the stresses were most critical, it should be possible to determine the critical point in the beam by computing for the inclined cracking load the stresses acting in the uncracked web along the trajectory of the actual web-shear crack. In Beams C.13.23, CD.13.24, and CD.13.25, the maximum principal tensile stress acting across the line of the crack was at the elastic centroid. In CD.14.34, the maximum principal tensile stress was at the junction of the web and lower flange and was 1.13 times the stress at the centroid. In similar computations for beams with straight wires,* the average ratio of the maximum principal tensile stress to that at the centroid was 1.04. In beams with short shear spans and thin webs, and cross sections similar to those tested, the maximum principal tensile stress

*The beams included in these computations were C.13.23 from this series; C.12.32 through C.12.57, C.22.40, and C.22.73 reported in Reference 1; CW.13.28, CW.13.38, CW.14.23, CW.14.25, CW.14.35 through CW.14.39, and CW.14.42 through CW.14.50 reported in Reference 2; C.10.27, C.10.28, CW.10.26, and CW.10.27 reported in Reference 3.

along the potential inclined crack occurs at the centroid. As the shear span gets longer or the web thickness increases, flexural stresses have a greater effect on the magnitude of the principal tensile stresses and the point of maximum stress along the potential web-shear crack tends to move toward the tension flange. At the same time, however, the properties of the beam become such that flexural cracks develop in the shear span before inclined cracking, and the computed stresses in the uncracked web cease to have any significance. Thus, in beams similar to those tested, it can be assumed that the web-shear cracking load can be determined by considering only the principal tensile stresses at the elastic centroid of the uncracked section.

Although no test data giving directly the tensile strength of the concrete are available, this quantity can be estimated on the basis of modulus of rupture tests. The true tensile strength of concrete would be the same as that measured in a beam test if the stress-strain curve in tension were linear all the way to rupture, which is probably nearly true for high strength concretes. Actually, the tensile strength of the concrete is less than the conventional modulus of rupture because the tensile stress-strain curve is nonlinear. The actual tensile strength of concrete lies somewhere between 60 and 100 percent of the modulus of rupture depending on the degree of nonlinearity of the tensile stress-strain curve.

For the purpose of predicting the type of inclined cracking in the test beams described in this paper, it was assumed that the tensile strength of the concrete was 80 percent of the modulus of rupture as computed by Eq. (1). Using this value of the tensile strength of the concrete, the web-shear cracking loads were computed for 30 beams which developed web-shear cracks.* The average ratio of observed to computed web-shear cracking load was 1.01 with a mean deviation of 0.10 and an over-all range of 0.70 to 1.21. Therefore, the web-shear cracking loads of beams similar to those tested can be computed by determining the shear required to raise the principal tensile stresses at the elastic centroid to 80 percent of the modulus of rupture.

In beams with shear spans shorter than about twice the depth, shearing forces tend to be transmitted to the supports by arch action rather than by beam action and the apparent tensile strength may be high if no allowance is made for the vertical "bearing" stresses. For shear spans greater than twice the depth, however, bearing stresses become insignificant in the vicinity of the crack and the apparent tensile strength should be approximately a constant fraction of the modulus of rupture of the concrete in the web.

For general application, a lower limiting value of the tensile strength should be used. It is recommended that the value of the tensile strength

*The beams included are those listed in the footnote on p. 664.

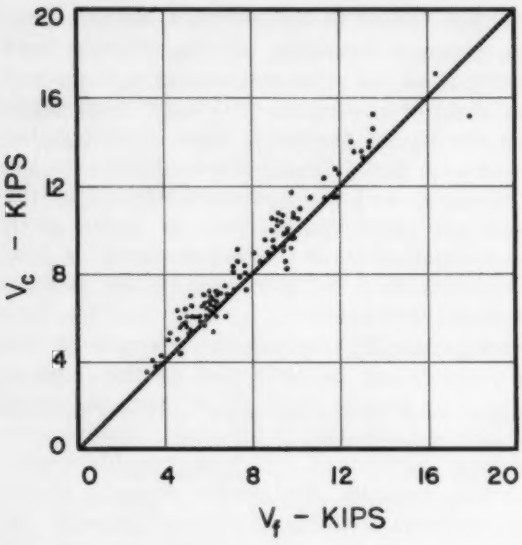

Fig. 10—Comparison of the measured inclined cracking shear and the computed shear corresponding to initiating cracks

of the concrete be taken as 1/2 to 2/3 the modulus of rupture rather than 80 percent as used in the analysis of these tests which were made under laboratory conditions.

Flexure-shear cracking—An inclined crack which develops after flexural cracks have formed in its vicinity in the shear span is defined in this paper as a flexure-shear crack. When a flexural crack forms in the shear span of a beam, it acts as a stress raiser and forces a redistribution of the principal tensile stresses in the uncracked part of the beam above or adjacent to it. In many cases, this redistribution of the stresses is such that the inclined crack forms at a load equal to or only slightly larger than the load causing the critical flexural crack in the shear span. In other cases, depending on the properties of the prestressed beam, the inclined cracking load may be as much as 20 to 30 percent greater than the load which causes the critical flexural crack.

In this paper, the flexural crack which affects critically the inclined cracking load is called the initiating crack. Its location is influenced by such factors as the length of the shear span and the relative magnitudes of the flexural cracking load and the web-shear cracking load. By examining the crack patterns for 109 beams with straight wires[1-3] which developed flexure-shear cracks, it was found that the initiating flexural crack generally started at the bottom of the beam at a distance from the load point ranging from one-half the height of the beam ($h/2$) to one-third the shear span ($a/3$). This distance approached $h/2$ as the load causing the initiating crack approached the computed web-shear cracking load, and approached $a/3$ if the load causing the initiating

crack was small compared to the web-shear cracking load. In all subsequent computations of the loads required to cause the initiating crack, the distance x from the load point to the critical initiating crack has been assumed to be the average of these two quantities as represented by the following equation:

$$x = a/6 + h/4 \quad \text{.....(2)}$$

Fig. 10 shows the relationship between the loads causing the initiating flexural cracks and the inclined cracking loads of the beams developing flexure shear cracks. In this figure, the ordinate of each point is the applied shear at inclined cracking, V_c, and the abscissa is the shear V_f at the computed load required to cause a flexural crack at a distance x from the load point. The majority of the plotted points lie above a 45-deg line since the inclined cracking loads should be greater than the load assumed necessary to cause the initiating crack.

Fig. 10 does not include any data for beams with web-shear cracks. If the load required to cause an initiating crack exceeds the web-shear cracking load, a web-shear crack will develop before a flexure-shear crack. In this case, the inclined cracking load is not related to the initiating flexural crack. Fig. 10 can be changed to the more general form shown in Fig. 11 and 12 by dividing both the abscissas and ordi-

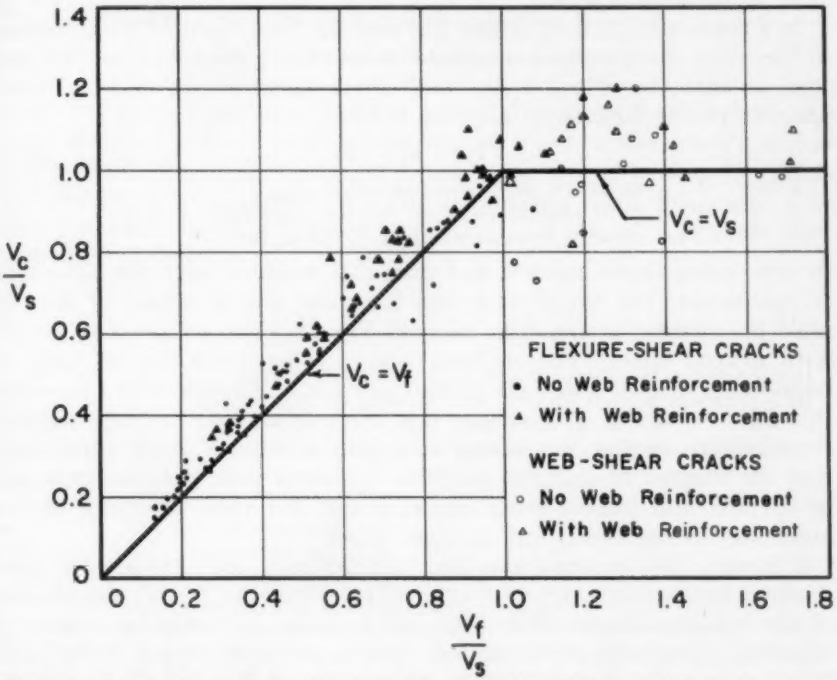

Fig. 11—Comparison of the dimensionless quantities V_c/V_s and V_f/V_s for beams with straight wires

nates of Fig. 10 by the shear V_s, corresponding to web-shear cracking, computed for an uncracked section as previously discussed. The results of 139 tests of prestressed concrete beams with *straight* longitudinal wires[1-3] are plotted in this manner in Fig. 11.

When the shear V_f corresponding to the initiating crack is less than that for a web-shear crack, V_s, the inclined cracking load is related to V_f. This relation is represented by the 45-deg line in Fig. 11. For V_f greater than V_s, the inclined crack should theoretically be a web-shear crack and V_c should equal V_s as indicated by the horizontal line in Fig. 11. The relation of the test data to the two lines plotted in Fig. 11 appears reasonable. The test results for beams which developed web-shear cracks fall along the line $V_c/V_s = 1.0$ while the test results for beams developing flexure-shear cracks fall in a band parallel to, but slightly above the 45-deg line representing $V_c = V_f$.

Effect of draped reinforcement on inclined cracking load of test beams

Flexure-shear cracking—The close relation between flexural cracking and inclined cracking is important in the interpretation of the behavior of beams with draped wires because, in the practical ranges of the variables, draped beams are usually susceptible to the flexure-shear type of inclined cracking.

In a beam with draped cables, the upward component of the prestress in the wires counteracts some of the shear due to dead load and applied load so that the effective or "net" shear force acting in each shear span at inclined cracking may be stated as:

$$V_n = V_e - V_d \quad \text{.....(3)}$$

where: V_n = net shear at inclined cracking
V_e = total applied shear at inclined cracking
V_d = upward component of prestressing force

In computing shear stresses and principal stresses, only the net shear is considered. For this reason, the ordinates and abscissas of Fig. 11 must be changed to the form used in Fig. 12 when considering beams with draped wires. The inclined cracking loads for the 21 tests of beams with draped wires are plotted in Fig. 12. The similarity between this figure and Fig. 11 indicates that the mechanisms causing inclined cracking are similar for beams with and without draped wires, and that the manner of inclined cracking of beams with draped wires can be divided into flexure-shear cracking and web-shear cracking in the same way as for beam with straight wires.

Although the inclined cracking mechanisms are similar for prestressed beams with and without draped wires, the actual magnitudes of the cracking loads differ since the moment required to cause the initiating flexural crack is smaller in a beam with draped wires than in a beam with straight wires. The effect of draping the wires in prestressed concrete beams can be shown by comparing the inclined

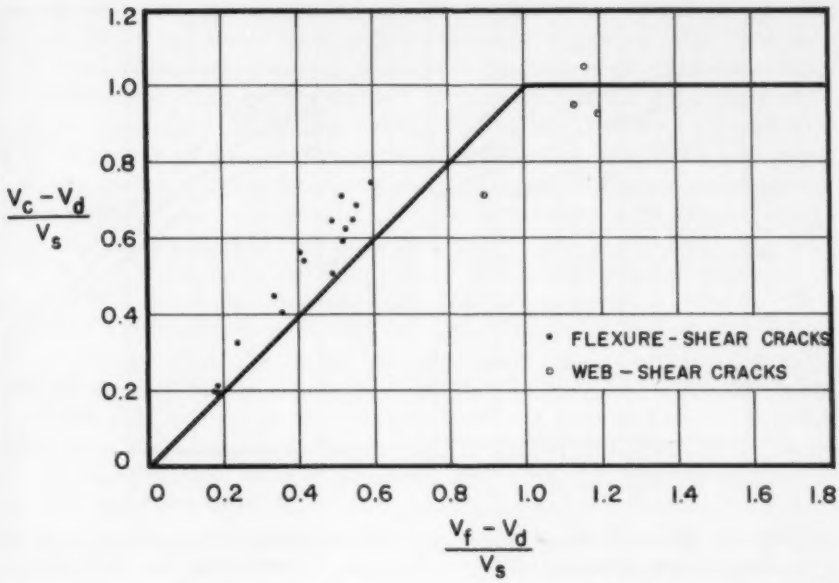

Fig. 12—Comparison of the dimensionless quantities $\frac{V_c - V_d}{V_s}$ and $\frac{V_f - V_d}{V_s}$ for beams with draped wires

cracking loads measured in these tests with draped wires with the corresponding loads for beams with straight wires. Since exactly similar beams with draped and straight wires were not available for direct comparison in every case, the inclined cracking loads for beams with straight wires were computed from Eq. (4) which is taken from Reference 1.

$$\frac{M_c}{(2f_r/3)\, bd^2 \sqrt{b'/b}} = 1 + \frac{F_{se}}{A_c(2f_r/3)} \qquad (4)$$

where: M_c = moment at inclined tension cracking defined as the product of the applied shear at the inclined tension cracking load and the length of the shear span, in.-lb
f_r = assumed modulus of rupture of concrete as defined by Eq. (1), psi
b = top flange width, in.
b' = web thickness, in.
d = effective depth, in.
F_{se} = effective prestress force, lb
A_c = gross area of cross section, sq in.

Since this expression gave results in excellent agreement with the results of 99 tests of beams similar in most respects to those tested with draped wires, it can be assumed to provide a satisfactory basis

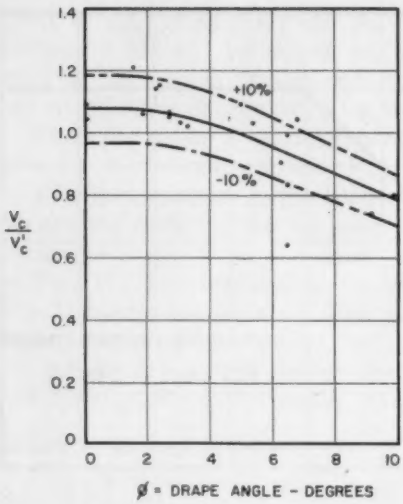

Fig. 13 — Effect of drape angle on flexure-shear cracking load

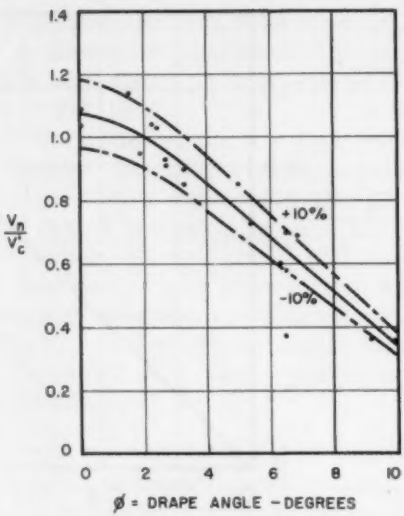

Fig. 14—Effect of drape angle on net shear corresponding to flexure-shear cracking

for comparison. Since Eq. (4) was derived to apply to flexure-shear cracks, it will only be used to compare beams developing such cracks.

The ratio of the measured inclined cracking shear for the beams included in these tests to the inclined cracking shear in an identical beam with straight wires, computed from Eq. (4), is plotted against the angle of drape in Fig. 13 for the 19 beams which developed flexure-shear cracks. The corresponding shears are listed in Table 4.

The test results in Fig. 13 indicate a reduction in the flexure-shear cracking load as the drape angle of the longitudinal reinforcement is increased beyond 2 to 3 deg.

It should be noted that the inclined cracking shear for the beams with draped wires in Fig. 13 is the total applied shear at a section. A similar plot based on the net shear as defined by Eq. (3) is shown in Fig. 14. Such a plot isolates the effects of the steel position on the cracking strength of the section, and the data in Fig. 14 emphasize the marked reduction in the net shear strength for draped beams which develop flexure-shear cracks. When the prestressing wires in the test beams were draped through the upper kern point at the support ($\phi = 10$ deg), the resulting net shear strength was only 30 percent of that for a beam with straight wires.

The considerable reduction in flexure-shear cracking load for the beams with draped wires can be explained in terms of the reduction in the load required to produce the initiating flexural crack. In the

beams with draped wires, the eccentricity of the reinforcement was decreased in the shear span and the prestressing force was thus less effective in resisting flexural tension at the bottom of the beam. Consequently, initiating cracks formed at lower loads in the beams with draped wires than in similar beams with straight wires. In addition, the increment of load between the formation of the initiating crack and the inclined crack was decreased since an initiating crack grows more rapidly and to a greater height in a beam with draped wires because the centroid of the steel is higher.

The development of splitting along the reinforcement adjacent to an inclined crack is adversely affected by draping the wires. This splitting results both from the doweling action of the steel crossing the crack and from the large change in steel stress which occurs adjacent to an inclined crack, as the behavior of the portion of the beam between the crack and the support tends to revert from "tied-arch action" to "beam action." The change in the tensile force must "flow" to the compression flange. This requires that, for the restoration of beam action, not only must there be adequate bond between the concrete and the steel, but the concrete section itself must be able to carry the shear flow from the tension to the compression flange. In a

TABLE 4 — SHEAR AT INCLINED CRACKING LOAD

Mark	Cracking shear		$\frac{V_c}{V_c'}$	Drape shear $A_s f_{se} \sin \phi$, V_d, kips	Net shear V_n, kips	$\frac{V_n}{V_c'}$	Drape angle, deg
	Computed V_c',* kips	Observed V_c,† kips					
AD.14.37a	10.20	6.55	0.64	2.75	3.80	0.37	6.45
AD.14.37b	11.48	9.45	0.83	2.75	6.70	0.58	6.45
B.14.34	7.78	8.49	1.09	0	8.49	1.09	0
B.14.41	8.85	9.21	1.04	0	9.21	1.04	0
BD.14.18	10.25	10.85	1.06	1.31	9.54	0.93	2.70
BD.14.19	10.20	11.16	1.09	2.45	8.71	0.85	5.00
BD.14.23	7.55	5.60	0.74	2.86	2.74	0.36	9.13
BD.14.26	7.95	6.38	0.80	3.63	2.75	0.35	9.95
BD.14.27	7.88	8.95	1.14	0.78	8.17	1.04	2.22
BD.14.28	8.25	—	1.21+	0.57	—	1.14+	1.53
BD.14.34	7.46	7.90	1.06	0.79	7.11	0.95	1.88
BD.14.35	7.16	6.49	0.905	2.14	4.35	0.61	6.28
BD.14.42	8.65	9.95	1.15	1.07	8.88	1.03	2.38
BD.24.32	7.54	7.45	0.99	2.20	5.25	0.70	6.45
BV.14.30	10.00	10.20	1.02	1.67	8.53	0.85	3.25
BV.14.32	10.02	10.37	1.03	1.34	9.03	0.90	3.25
BV.14.34	10.02	10.48	1.05	1.36	9.12	0.91	2.70
BV.14.35	9.50	9.80	1.03	2.84	6.96	0.73	5.37
BV.14.42	9.20	9.60	1.04	3.29	6.31	0.69	6.81
C.13.23a	8.26	10.0	1.21	—	10.0	1.21	0
C.13.23b	7.91	6.61	0.84	—	6.61	0.84	0
CD.13.24a	8.40	10.69	1.27	1.34	9.35	1.11	3.40
CD.13.24b	7.93	9.10	1.15	1.34	7.76	0.98	3.40
CD.13.25	7.95	9.89	1.24	1.11	8.78	1.10	2.85
CD.14.34	5.85	5.45	0.93	0.74	4.71	0.81	1.88

*V_c' is from Eq. (4) for all the beams with flexure-shear cracks and from principal tensile stresses for the beams with web-shear cracks.

†All the AD, B, BD, and BV series beams developed flexure-shear cracks except BD.14.28 which did not develop inclined cracks.

All the C and CD series beams developed web-shear cracks.

beam with straight wires, the web to flange juncture is the critical section in this transfer of shearing forces. In a beam with draped wires, the presence of wires in the web introduces another plane of weakness along which splitting can develop. Furthermore, the change in steel stress adjacent to an inclined crack tends to be higher in a beam with draped wires because, at an uncracked section, the tensile strains in the reinforcement are smaller the closer the reinforcement is to the neutral axis of the beam. Thus, in a beam with draped wires there is a greater variation in steel stress between a cracked section and an adjacent uncracked section than in a comparable beam with straight wires. This increased stress difference results in a large shear flow adjacent to the crack which may cause splitting along the wires.

Web-shear cracking — Theoretically, the web-shear cracking load of a prestressed concrete beam will be increased by draping the wires since the upward component of prestress offsets a portion of the dead load and live load shears and thus reduces the principal tensile stresses in the web. Beams CD.13.24 and CD.13.25 were tested to check the validity of this concept. Beam CD.13.24 was tested with a load first at one end and then at the other, giving two values of the web-shear cracking load. The data from these two tests and the test on CD.13.25 are plotted in Fig. 15 along with the web-shear cracking data obtained from tests of beams with straight wires. Fig. 15 (a) is plotted for the gross shear, while Fig. 15(b) is plotted for the net shear. The points plotted at $\phi = 0$ compare the measured and computed web-shear cracking loads for 30 beams with straight wires and are the same on both figures since the gross and net shear are the same in a beam with straight wires. (Beam CD.14.34 which is also plotted in Fig. 15 will be discussed later.)

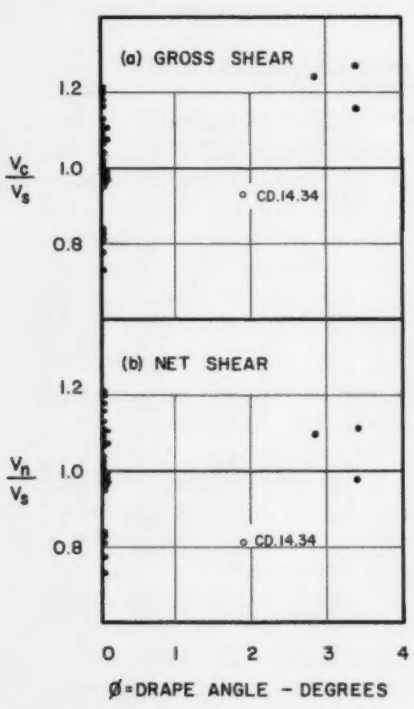

Fig. 15—Effect of drape angle on shear corresponding to web-shear cracking

From Fig. 15(a) it appears that the gross shear at web-shear cracking is increased by draping the wires although the three points for Beams CD.13.24 and CD.13.25 all fall within the range of the data for beams with straight wires. If this figure is replotted in terms of

the net as shown in Fig. 15(b), the test data for the draped beams agree more closely with the average for the beams with straight wires. From these two figures, it appears that the inclined cracking strength of beams developing web-shear cracks is increased by draping the wires. Since flexural cracks do not influence the development of this type of crack, the "net shear capacity" of beams developing web-shear cracks depends only on the principal stresses in the web and, therefore, remains constant as the drape angle increases.

If the draped wires enter the web in the shear span, however, the stress concentrations introduced by shear transfer at the level of the steel or by the reduction of the concrete section may be sufficient to raise the principal tensile stresses at that location to a critical value. This condition did not exist in Beams CD.13.24 and CD.13.25 since the draped wires did not enter the web. However, in CD.14.34 the web-shear crack originated approximately at the point where the wires entered the web (see Fig. 9), and the load at inclined cracking was only 81 percent of the predicted value.

Strength of prestressed beams with draped wires

Beams without web reinforcement — The ultimate load for the beams without web reinforcement was either equal to or only slightly greater than the inclined cracking load as shown in Fig. 16. The tests plotted for $\phi = 0$ deg include B.14.34, B.14.41, and all the similar beams reported in Reference 1. The beams with low drape angles and the beam with the low prestress level (BD.24.32) carried some additional load beyond inclined cracking but the increase was still small, and the "useful capacity" of the beam corresponded closely to the inclined cracking load. Moreover, except where the prestress level was relatively small, the beams with the higher drape angles failed at the inclined cracking load.

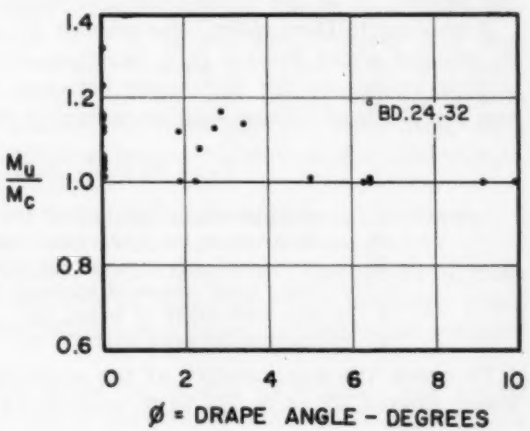

Fig. 16 — Increase in load after inclined cracking for test beams without web reinforcement

TABLE 5 — MEASURED AND COMPUTED MOMENTS

Mark	Observed inclined cracking moment M_c, in.-kip	Observed ultimate moment M_u, in.-kip	Flexural moment capacity M_{uf}, in.-kip	Moment capacity shear failure* M_{us}, in.-kip	$\frac{M_u}{M_c}$	$\frac{M_u}{M_{uf}}$	$\frac{M_u}{M_{us}}$	Mode of failure†
AD.14.37a	235	235	481	—	1.00	0.49	—	S
AD.14.37b	303	306	481	—	1.01	0.64	—	S
B.14.34	305	322	376	—	1.06	0.81	—	S
B.14.41	332	348	475	—	1.05	0.68	—	S
BD.14.18	391	442	544	—	1.13	0.81	—	S
BD.14.19	402	404	558	—	1.00	0.72	—	S
BD.14.23	202	202	400	—	1.00	0.50	—	S
BD.14.26	230	230	385	—	1.00	0.60	—	S
BD.14.27	358	358	385	—	1.00	0.93	—	S
BD.14.28	—	360	389	—	—	0.93	—	F
BD.14.34	294	318	382	—	1.12	0.83	—	S
BD.14.35	234	234	380	—	1.00	0.62	—	S
BD.14.42	322	348	457	—	1.08	0.76	—	S
BD.24.32	268	320	483	—	1.19	0.66	—	S
BV.14.30	368	443	497	498	1.21	0.74	0.74	SB
BV.14.32a	374	455	515	520	1.21	0.88	0.87	SB
BV.14.32b	—	530	526	520	—	1.00	1.02	F
BV.14.34	368	464	494	524	1.26	0.94	0.85	F
BV.14.35	353	455	482	542	1.29	0.95	0.84	F
BV.14.42	353	444	423	446	1.26	1.05	1.00	F
C.13.23b	178	208	433	—	1.16	0.48	—	S
CD.13.24b	246	280	441	—	1.14	0.63	—	S
CD.13.25	267	315	431	—	1.17	0.73	—	S
CD.14.34	196	196	380	—	1.00	0.52	—	S

*Tabulated only for beams with stirrups.
†S, F, and SB refer to shear, flexure, and secondary bond failures, respectively.

Beams with web reinforcement — When sufficient web reinforcement was added to beams with draped wires, the mode of failure changed from shear to flexure. The inclined cracking load was not greatly affected by the presence of stirrups but the ductility of the beam after cracking was improved. The web reinforcement provided a means of transferring shear force across the crack and tended to restrain rotation at the cracked section. In addition, the development of splitting along the tension reinforcement was restrained, preventing failures due to the separation of the wires from the beam.

According to Hernandez,[2] the amount of web reinforcement required to prevent shear failure in a prestressed beam with straight cables is proportional to the difference between the inclined cracking load and the flexural failure load as shown in the following equation.

$$V_u - V_c = \frac{5}{4} A_v f_{vy} \frac{d}{s} \quad \text{(5)}$$

where: V_u = ultimate shear capacity of the section, kips
V_c = shear at inclined cracking load, kips
A_v = cross-sectional area of one stirrup, sq in.
f_{vy} = yield point stress of stirrups, ksi
d = effective depth of beam, in.
s = spacing between stirrups, in.

To check the applicability of the expression to beams with draped wires, Beams BV.14.30, BV.14.32, and BV.14.42 were provided with an

amount of web reinforcement computed from Eq. (5) to be just sufficient to produce a balanced shear and flexural failure. In addition, Beams BV.14.34 and BV.14.35 were provided with 1.34 and 1.85 times the amount required for balanced failures. Both of the latter beams failed in flexure as expected, as did Beams BV.14.32b and BV.14.42 which had balanced design. Beams BV.14.32 and BV.14.30 with the balanced design failed at less than their flexural capacity but this was due to a secondary failure in bond in the anchorages after inclined cracking. The bond strength of the draped wires might have been lower with respect to a beam with straight wires due to the settlement of a larger depth of concrete underneath the wires. It appears, therefore, that if the anchorage bond is adequate, Eq. (5) can be used to predict the additional strength provided by stirrups in a beam with draped wires and web reinforcement if the cracking load shear includes an allowance for the effect of draping the wires. Since the inclined cracking load is reduced by draping the wires in beams developing flexure-shear cracks, more web reinforcement is required to ensure a flexural failure in this type of beam with draped wires than is required in a similar beam with straight wires.

SUMMARY

The object of this investigation was to determine the effect of linearly-draped reinforcement on the shear strength of prestressed concrete beams. Tests of 22 pretensioned beams with varying degrees of drape are described in this report. The results of these tests are compared with those of 139 other tests of similar beams with straight reinforcement, reported in References 1, 2, and 3.

Twenty-one of the beams tested were I-beams; the remaining beam was rectangular in cross section. The concrete strength varied from 2560 to 6280 psi, the longitudinal steel ratio was either 0.3 or 0.4 percent, and the effective prestress was about 110,000 psi. All beams had overall cross-sectional dimensions of 6 x 12 in. The I-beams had web thickness of 1¾ or 3 in. Five beams were provided with web reinforcement. The principal variable in the test series was the profile of the prestressing steel. Twelve different profiles were used, each consisting entirely of straight line segments bent at the load points, as indicated in Fig. 2 and Table 2.

Nineteen of the beams were tested to failure with equal loads applied at the third points of a 9-ft span. The remaining three beams, all of which had thin webs, were tested with a single load applied 27 in. from the end of a 102-in. span. Records of load, deflection, concrete strain at the top of the beam, steel strain, and crack pattern were obtained at all stages of loading.

The majority of the beams failed in "shear" as a direct and almost immediate consequence of the formation of inclined tension cracks

which affected detrimentally both the strength and the ductility of the beams. All other variables being equal, draping the longitudinal reinforcement in beams of practical proportions resulted in a lower inclined cracking load and, therefore, a weaker beam.

Because of the different phenomena involved in their initiation, the inclined tension cracks were classified into two types: flexure-shear cracks which developed adjacent to or as an extension of flexural cracks in a region of combined bending and shear; and web-shear cracks which originated in the web before any flexural cracking in their vicinity.

The load corresponding to the formation of a flexure-shear crack was found to be closely related to the flexural cracking load near the point where the flexure-shear crack originated. On the other hand, the load corresponding to the formation of a web-shear crack could be computed on the basis of an uncracked section analysis. Web-shear cracks occurred in beams having very thin webs, high ratios of shear to maximum moment, and high prestresses. Flexure-shear cracks prevailed in beams having more practical proportions.

As the angle of inclination of the prestressing steel was increased, there was a reduction in the flexure-shear cracking load, although at low drape angles, the reduction was relatively small. However, in no case could it be said conclusively that draping the reinforcement increased the inclined cracking load of beams developing flexure-shear cracks. On the other hand, the limited data available for beams developing web-shear cracks indicated that draping the wires increased the web-shear cracking load. It should be emphasized, however, that flexure-shear cracking is more critical than web-shear cracking in practical structures such as bridges.

Failure in shear occurred by shear-compression, by separation of the tension reinforcement from the beam caused by splitting along the wires, and by distortion and crushing of the web. Generally, the beams with high drape angles failed suddenly at the inclined cracking load. The other beams carried up to 19 percent additional load after inclined cracking.

The limited tests on beams with draped wires and web reinforcement indicated that stirrups could be used to prevent shear failures in beams with draped wires. The presence of web reinforcement did not affect the inclined cracking load. However, because the inclined cracking load decreased as the drape angle was increased, more web reinforcement was required to provide a flexural failure in beams with high drape angles than in beams with low drape angles.

In general, it was concluded that draping the longitudinal wires did not increase either the inclined cracking load or the ultimate shear strength of prestressed concrete beams which developed flexure-shear cracks. Instead, the trend of the test results indicated a reduction in

both the inclined cracking load and the ultimate strength of these beams. In the case of the three beams which developed web-shear cracking, however, draping the wires increased the inclined cracking load. Both types of inclined cracking could occur in beams of practical proportions.

ACKNOWLEDGMENTS

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Title No. 57-32

Static Modulus of Elasticity of Concrete as Affected by Density

By ADRIAN PAUW

The elastic modulus of concrete is an important parameter in reinforced concrete design and analysis. With the increased use of lightweight aggregates for structural concrete a better understanding of the relationship between weight, strength, and the elastic modulus is needed. In this study the static modulus for a large variety of aggregates and concrete strengths was analyzed and an empirical formula was derived which is applicable to both lightweight and normal weight structural concretes. The formula is in excellent agreement with recognized empirical formulas for normal weight concrete.

■ IN REINFORCED CONCRETE and especially in prestressed concrete a knowledge of strains as well as of stresses is important. Strains may be classified into four types: elastic strains, lateral strains, creep strains, and shrinkage. This paper is limited to a consideration of elastic strains.

The term elastic strain when applied to concrete is somewhat ambiguous since the stress-strain curve for concrete is seldom a straight line, even at normal working stress levels; and, furthermore, the strains for the first loading cycle are seldom entirely recoverable. If creep strains are eliminated from consideration, the lower portion of the instantaneous stress-strain curve is usually found to be relatively straight after the specimen is subjected to an initial load cycle resulting in a compressive load equivalent to about one-half the compressive strength. The slope of this curve may be considered as the static modulus of elasticity. This modulus varies with such factors as strength of concrete, age of concrete, properties of aggregates and cement, rate of loading, and type and size of specimen, as well as of the definition of the term modulus of elasticity itself, whether initial, tangent, or secant modulus.

ELASTIC MODULUS OF NORMAL WEIGHT CONCRETE

To date no standard test has been adopted for the determination of the static modulus of elasticity for concrete, thus further adding to the ambiguity of the term. Several empirical formulas have previously been proposed to serve as a guide for use when the modulus cannot be determined by test. Examples of these follow.

1. ACI Committee 318 ACI Building Code formula:

$$E_c = 1000\ f_c' \quad (1)$$

where E_c = modulus of elasticity, psi
f_c' = 28-day compressive strength

2. ACI-ASCE Committee 323 formula:

$$E_c = 1{,}800{,}000 + 500\ f_c' \quad (2)$$

3. Empirical formula proposed by Jensen:[1]

$$E_c = \frac{6 \times 10^6}{1 + 2000/f_c'} \quad (3)$$

4. Empirical formula developed by Inge Lyse:[2]

$$E_c = 1{,}800{,}000 + 460\ f_c' \quad (4)$$

All of the above formulas are applicable only to normal weight concretes. Eq. (1) is a simple approximation which is reasonably accurate for concretes having a compressive strength of about 3000 psi, but for

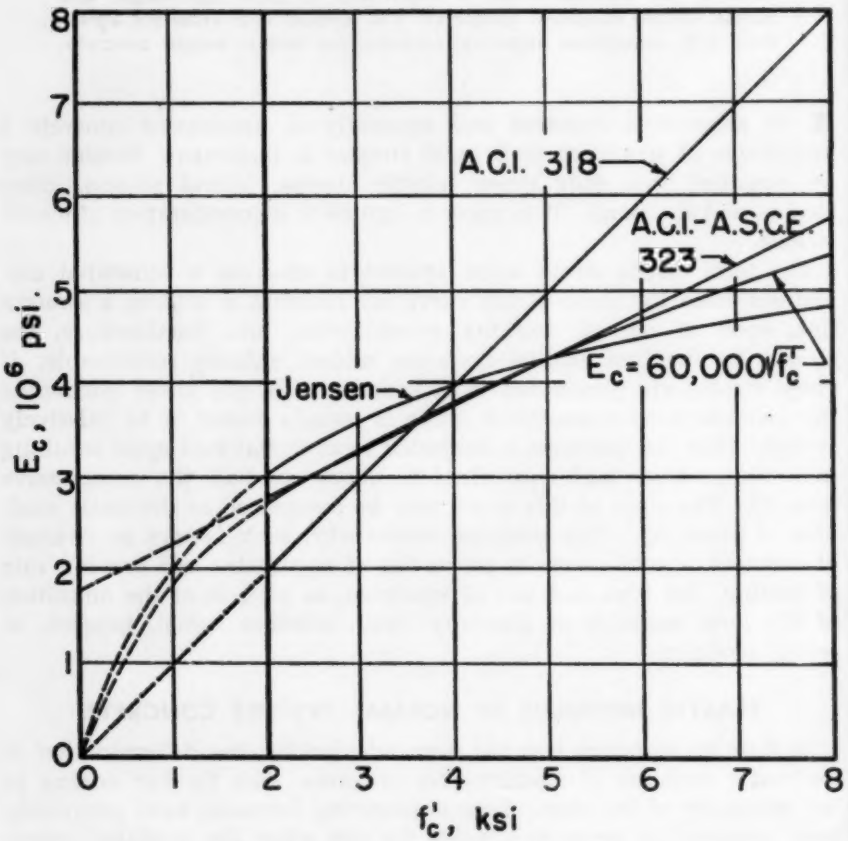

Fig. 1—Empirical formulas for E_c

ACI member **Adrian Pauw**, professor of civil engineering, University of Missouri, Columbia, is now on leave from the university as a Fulbright lecturer at Delft Technological Institute, Delft, Netherlands. He is chairman of ACI Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete, and a member of Committee 337, Strength Evaluation of Existing Concrete Structures. Professor Pauw's areas of special interest, in addition to lightweight aggregate concrete, include structural theory and machinery foundations.

higher strength concretes this formula yields values for E_c which may be as much as 50 percent too high. Eq. (2), (3), and (4) yield essentially similar values for concrete strengths ranging from 2000 to 7000 psi but their applicability is limited to normal weight concrete. The basic form of Eq. (2) and (4) limits their applicability to a specific range of compressive strength values since for very low values of f_c' these formulas yield values of E_c greater than 1,800,000 psi. Similarly the formula proposed by Jensen limits the maximum value of the elastic modulus to 6,000,000 psi.

With the rapid expansion of the use of lightweight aggregate concrete in recent years, the need for a suitable empirical relationship for the elastic modulus of these concretes has become increasingly apparent. An earlier study showed that the empirical relationship

$$E_c = 60{,}000 \sqrt{f_c'} \quad \text{.....(5)}$$

yields values for E_c within the limits of the values given by the formulas proposed by Lyse and Jensen for normal weight concretes having compressive strengths greater than 2000 psi as demonstrated by the plot in Fig. 1. This relatively simple formula has the further advantage of yielding reasonable results for concretes with both very low and very high compressive strengths.

PROPOSED EMPIRICAL FORMULA

It has been observed by many investigators that the elastic modulus of lightweight aggregate concretes is considerably lower than the values of normal weight concretes of comparable compressive strength and that the modulus appears to be a function of the weight. It is known that all mineral aggregates have about the same absolute specific gravity. The difference in weight of various types of concrete is therefore primarily the result of voids in the concrete, whether they be due to purposely entrained air, or due to the vessicules in lightweight aggregate. From these considerations it was suspected that it might be possible to obtain a satisfactory approximation by expressing the value of the elastic modulus by an empirical relationship of the form

$$E_c = a w^{3/2} \sqrt{f_c'} \quad \text{.....(6)}$$

where E_c = static modulus of elasticity of the concrete, psi
w = air-dry weight of the concrete at time of test, pcf
f_c' = compressive strength of the concrete at time of test, psi
a = a suitable constant

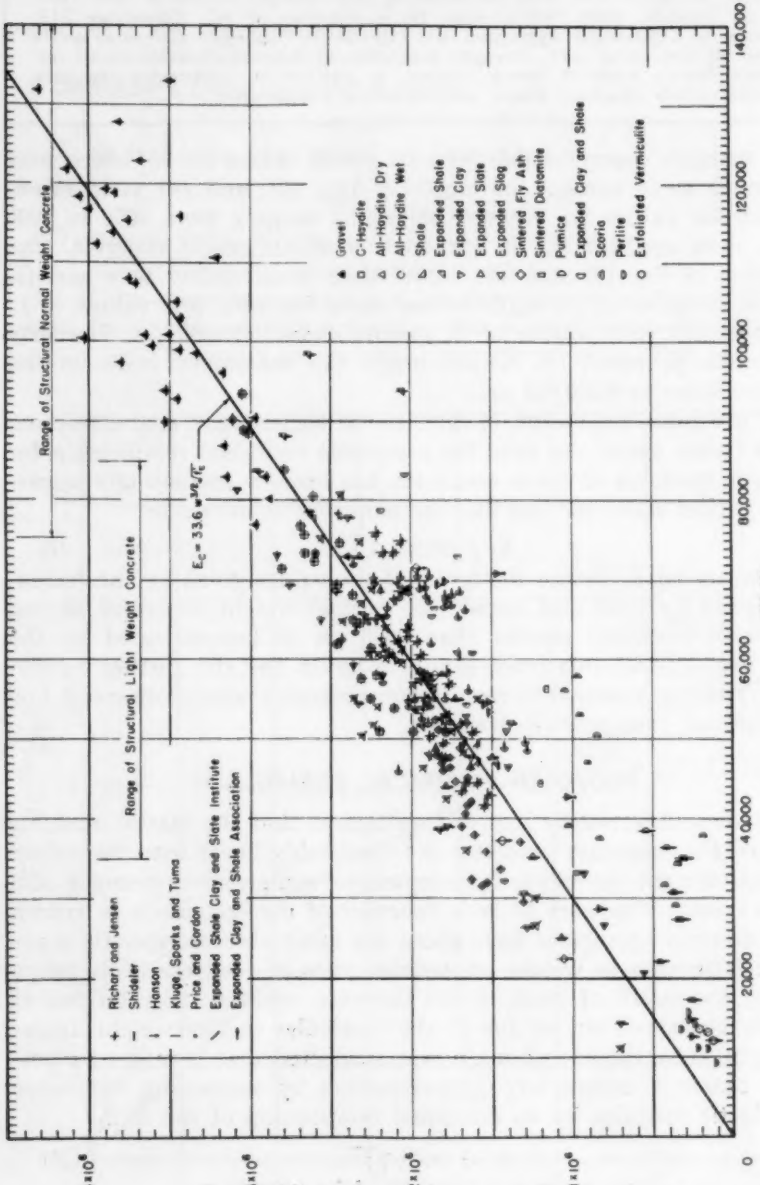

Fig. 2—Correlation of test data

TEST DATA

In searching the literature for data which would be useful in testing Eq. (6), several difficulties were encountered. The first difficulty was the result of the lack of a standard test for measuring elastic modulus—some investigators reporting the initial tangent modulus whereas others, a secant modulus at some given stress level. In some instances the method used to determine the static modulus was not even defined. Then it was found that in many cases the unit weight of the concrete at time of test was not reported and therefore such data could not be further considered. In a few instances only the wet weight of the concrete was reported; these data were used by adjusting the weight by applying reduction factors based on experience with similar aggregates. The reports in which suitable data were found are listed as references.

The most important source of information was the report of the work of Richart and Jensen.[3] Materials tested included gravel, C-Haydite, and all-Haydite, both dry and moist. Nine mixes were tested ranging from 1:1.5:2.5 to 1:3.5:2.5. The static modulus values reported are initial-tangent modulus values. Reported values of E_c ranged from 55 to 75 percent of the values obtained for gravel concrete of comparable strength.

The results of 32 tests were obtained from the work reported by Shideler.[4] The static modulus values reported, based on the secant modulus at about 0.3 f_c' range from 53 to 82 percent of the values for sand and gravel concrete of comparable strength. The aggregates tested included shales, slates, and clays expanded in rotary kilns as well as aggregates produced by the sintering process. Expanded blast-furnace slags were also tested. Materials tested were obtained from a wide range of geographical locations.

The results of 18 tests reported by Hanson[5] are also included. Only the weight of the concrete in the plastic condition was reported and these weights were therefore adjusted on the basis of information given in Reference 4.

The paper by Kluge, Sparks, and Tuma[6] furnished data on a variety of aggregates including exfoliated vermiculite, sintered diatomite, perlite, expanded blast-furnace slags, sintered fly ash, pumice, as well as expanded shale, slate and clay. The modulus of elasticity was determined by both static and dynamic test methods. The observed modulus of elasticity for some of the concretes produced with the weaker aggregates was found to be very low.

Data for aggregates produced in the western part of the United States were obtained from a paper by Price and Cordon.[7] Represented are two expanded shales and clays, an expanded slag, scoria, four samples of pumice, five samples of perlite, and two exfoliated vermiculites. Most of the weaker aggregates produced concretes having very low moduli of elasticity. All specimens were fog-cured for 7 days fol-

Fig. 2—Correlation of test data

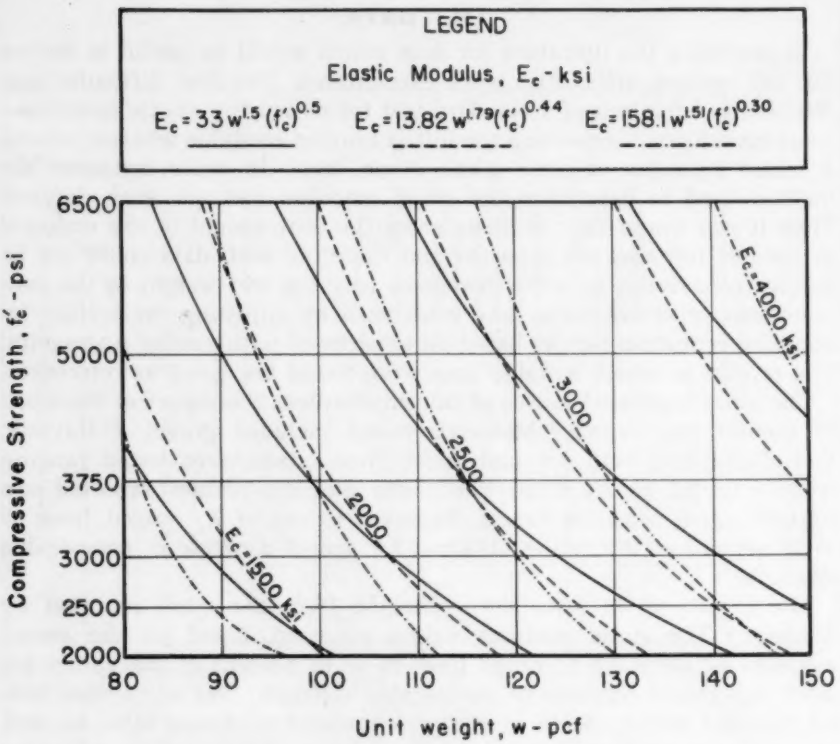

Fig. 3—Comparison of empirical formulas for E_c

lowed by 21 days of curing at 50 percent humidity. Tests results were for age 28 days and the static modulus values were reported. The basis used for determining the static modulus was not reported.

Additional data for concretes produced from expanded shales, slates, and clays were obtained from unpublished reports furnished by the Expanded Shale, Clay and Slate Institute.[8,9] Both sonic and secant modulus values were reported but only the latter were considered in this study.

ANALYSIS OF DATA

The value of the constant a in Eq. (6) was determined by considering the ratio $E_c/(w^{3/2}\sqrt{f_c'})$. Fig. 2 is a plot of E_c as a function of $w^{3/2}\sqrt{f_c'}$. Using the method of least squares the value of the constant a in Eq. (6) was found to be 32.43. Since most of the data reported by Price and Cordon were for concretes produced with relatively weak aggregates such as the pumices and perlites, the value of a was recomputed, omitting these data. The value of a on this basis was found to be 33.6.

This value is believed to be a reasonable compromise since most of the concretes tested by Price and Cordon cannot be considered to be of structural quality.* Thus for concretes of structural quality the static modulus of elasticity may be determined by the approximate empirical equation

$$E_c = 33\ w^{3/2}\ \sqrt{f_c'} \qquad (7)$$

After the computations for the constant a were completed some additional data were obtained from the Expanded Clay and Shale Association.[10] These data, for 16 different lightweight concrete mixes, have also been plotted in Fig. 2 but they were not included in the least-square analysis. It is believed however that inclusion of these data would not have produced any significant change in the final results.

Considering the limitations of the data available, the correlation between the reported elastic modulus and the values predicted by Eq. (7) appears rather remarkable. In view of the many other variables involved in concrete design it is believed that this equation is adequate for most design purposes. As a further check on the validity of the proposed formula, a generalized form of Eq. (6) was analyzed:

$$E_c = A\ w^B\ (f_c')^C \qquad (8)$$

By writing Eq. (8) in logarithmic form

$$\log E_c = \log A + B \log w + C \log f_c' \qquad (9)$$

a linearized equation was obtained which was susceptible to a non-orthogonal regression analysis. This analysis yielded the formula

$$E_c = 13.82\ w^{1.79}\ f_c^{0.44} \qquad (10)$$

with all the data included, and the formula

$$E_c = 158.1\ w^{1.31}\ f_c^{0.30} \qquad (11)$$

when the data for concretes having a compressive strength of less than 2000 psi were omitted. The exclusion of these data is believed to be justified since elastic modulus is generally of interest only in the case of structural quality concretes. A number of the weaker concretes were made with friable aggregates and such concretes are not suitable for structural work. It may be noted from Eq. (11) that the value of the coefficient B seems to confirm the predicted value used in Eq. (6), while the predicted value of the coefficient C appears to be somewhat high. However, considering the limitations of the data used, a further refinement of Eq. (7) does not appear to be warranted until new data determined on a standardized test basis becomes available. Fig. 3 shows the value of elastic modulus obtained by Eq. (10) and (11) as compared with the value obtained by the proposed equation. It may be noted that all these equations predict reasonably similar results and

*Structural quality concrete is here defined as a concrete having compressive strength of 2500 psi or higher.

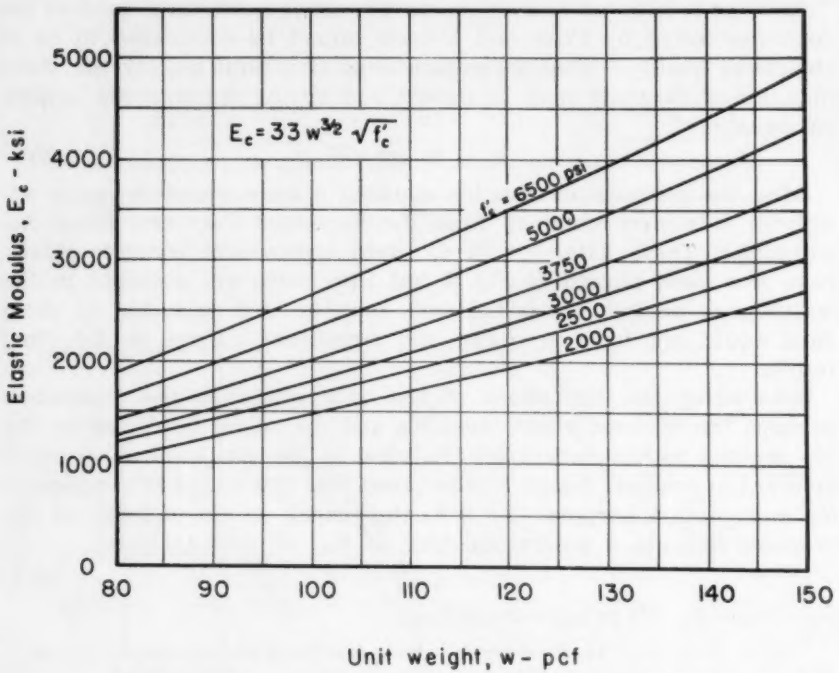

Fig. 4—Elastic modulus as a function of strength and weight of concrete

that the value of modulus is much more sensitive to small changes in the weight of the concrete than it is to the compressive strength.

SUMMARY AND CONCLUSIONS

The static modulus of elasticity of both normal and lightweight structural concrete may be approximately determined by the empirical formula

$$E_c = 33\, w^{3/2} \sqrt{f_c'}$$

The value of the elastic modulus is more dependent on the weight of the concrete and the method of test used to determine it than on the compressive strength of the concrete. The reliability of the proposed formula appears to be as good as recognized and acceptable empirical formulas for normal weight concretes. The proposed formula should therefore prove useful to the designer by providing a single uniform method for estimating the modular ratio for any structural quality concrete. A convenient graphical form of the relationship is given in Fig. 4.

ACKNOWLEDGMENTS

The assistance of Isaac Gordon, graduate research assistant, Department of Civil Engineering, and Roy F. Keller, acting director, University of Missouri Computer Center, in processing and analyzing the data is gratefully acknowledged. The author is indebted to Frank G. Erskine of the Expanded Shale, Clay and Slate Institute for his suggestion that the elastic modulus might be related to the weight and for furnishing unpublished test data. The author also wishes to express his appreciation for the data furnished by Lucas E. Pfeiffenberger of the Expanded Clay and Shale Association.

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Title No. 57-33

Reconsolidation Improves Grouted Masonry Wall Panels

By MANLEY W. SAHLBERG

The type of wall described is actually a reinforced concrete wall cast between bricks or masonry units which act as absorptive forms. It satisfies architects who like brick facings and it also satisfies engineers who want to use these walls to resist lateral forces induced by earthquakes.

The test described was started to demonstrate the practicability of high lift grouting of cavity masonry walls. Some panels were made without vibration, some used vibration or tamping, and others utilized vibration and revibration.

■ A PROJECT WAS UNDERTAKEN TO DEMONSTRATE the practicability of high lift grouting of masonry walls, i.e., the placing of the concrete grout core from bottom to top in a continuous operation in one day. Full scale panels, when properly cured a minimum of 28 days, were laid over (Fig. 1) in a horizontal position and then cut up (Fig. 2) with a masonry saw to observe results. Fig. 3 shows panels E and F still standing with panels A and C cut up in the foreground.

The panels were planned to be all 12 ft high and 8 ft long with wall thickness and makeup varied. Panels A to P, inclusive, were of this size except panels G, H, I, J, K-1, and K-2 which were reduced to 6 x 4 ft, pilot panel size to save cost.

The various masonry units and combinations used in the panels are noted in Table 1. Note that both ⅜-in. and ¾-in. maximum size gravels were used. The various consolidation and reconsolidation methods used for the grout concrete are listed.

CONSOLIDATION AND RECONSOLIDATION OF GROUT

Panels A to F were grouted purposely *without* vibration or tamping to gain knowledge of the results of "self" consolidation. Two major items were in mind; first, the resistance of the stirrup type ties to the high pressures of grout during rapid placement; and second, the results with various combinations and materials used for the masonry walls, which were in reality masonry forms.

Manley W. Sahlberg, late principal structural engineer of the San Francisco office of the Schoolhouse Section of the State Division of Architecture, area structural engineer for Area I comprising about a third of the state, was responsible for directing the checking of plans and supervising construction of public schools in the area from 1954 until his death in August, 1960. He rejoined the division in 1948 in Los Angeles after having served the division in 1934 and 1935 after the earthquake of 1933. During 1941 to 1945 he was a staff engineer for Donald R. Warren Co., consulting engineers, on the structural design of the steel plant at Fontana for Kaiser Co. He was in private practice in Sacramento in the 20's and in Los Angeles during the 30's. He was a member of ACI, ASCE, and the Structural Engineers Association of Northern California.

Panels A to F were successfully grouted at a rather rapid rate to full height with mechanized equipment, and proved the effectiveness of the stirrup type masonry form tie, spaced 12 in. vertically and 24 in. horizontally, to hold the masonry forms. Fig. 4 shows 12 ft high panels being grouted.

Cutting the panels into strips gave visual evidence of the "self" consolidation and the grout setting behavior for the full height of the wall panels. That concrete consolidation could be improved was obvious. Nevertheless, cores were taken from these panels and tested and would have qualified the panels for acceptance under prevailing codes.

TABLE 1 — PANEL MAKE-UP

Panel	Height, ft	Length, ft	Thickness, in.	Makeup of wythes	PG, in.	Slump, in.	V	RV
A	12	8	12	Brick—Economy unit	¾	10	NC	....
B	12	8	10	Brick, 1 face-Norman face Glazed tile-Structural	⅜	10	NC	....
C	12	8	9½	Brick—Jumbo	⅜	10	NC	....
D	12	8	8½	Brick—Jumbo	⅜	10	NC	....
E	12	8	12	Brick—Jumbo	¾	10	NC	....
F	12	8	12	Brick—Economy unit	¾	10	NC	....
G	6	4	9	Brick—Jumbo	⅜	10	IV	....
H	6	4	9	Brick—Jumbo	⅜	10	T	....
I	6	4	12	Brick—Economy unit	¾	7-10	IV	
J	6	4	12	Brick—Economy unit	¾	7-10	T	RT
K-1	6	4	9½	Brick—Jumbo, buff	⅜	11	IV	IRV
K-2	6	4	9½	Brick—Jumbo, buff	⅜	11	T	RT
L	12	8	9½	Brick (Pozzolan Grout)—Economy unit	⅜	11	T	RT
M	12	8	9½	Brick-1 face—Jumbo Split concrete block, 1 face	⅜	11	T	RT
N	12	8	9½	Brick—Economy units	⅜	11	SV	SRV
O	12	8	9½	Brick-1 face—Jumbo Split concrete block, 1 face	¾	11	T	RT
P	12	8	9½	Brick, 1 face—Economy unit Brick—Jumbo, buff	¾	11	SV	SRV

PG = Maximum size of pea gravel
V = Consolidation
RV = Reconsolidation
NC = No consolidation
IV = Internal vibration
T = Tamping or rodding
SV = Steel mat vibration
RT = Retamping or rerodding
IRV = Internal revibration
SRV = Steel mat revibration

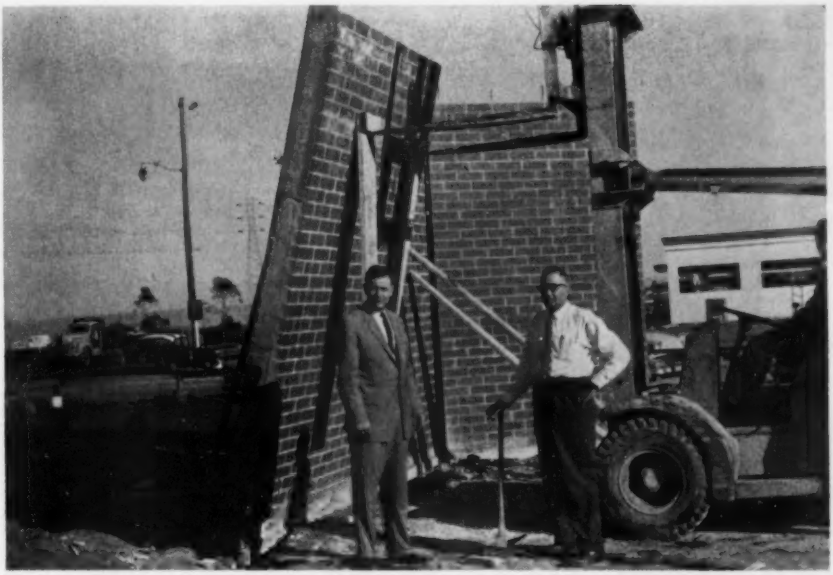

Fig. 1—Typical method of lowering panels to horizontal position with lift truck

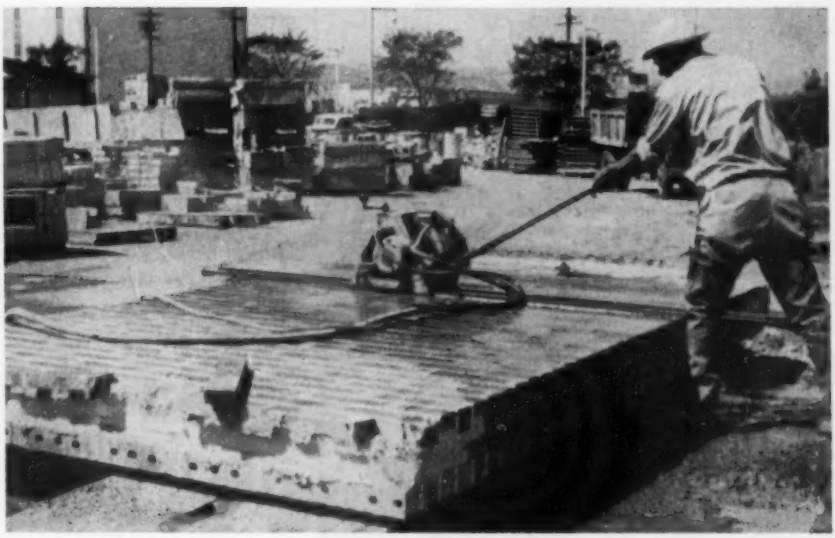

Fig. 2—Cutting panels into strips

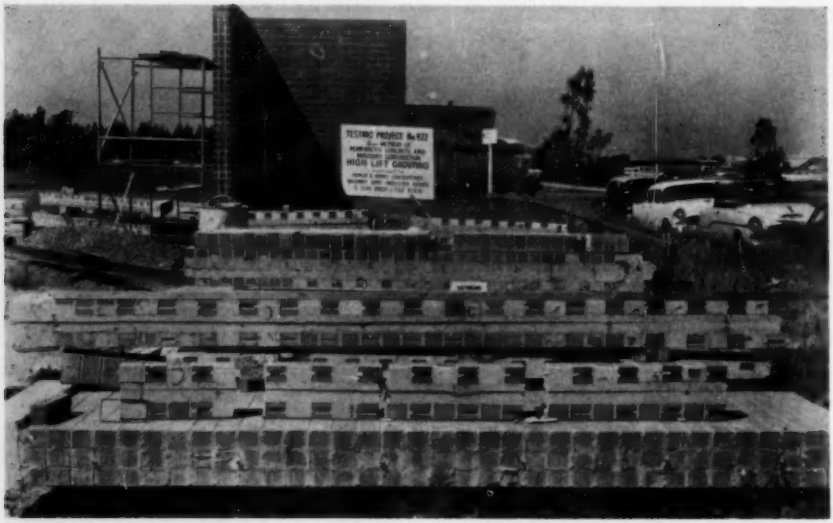

Fig. 3—Cut panels in foreground. Panels E and F standing in background

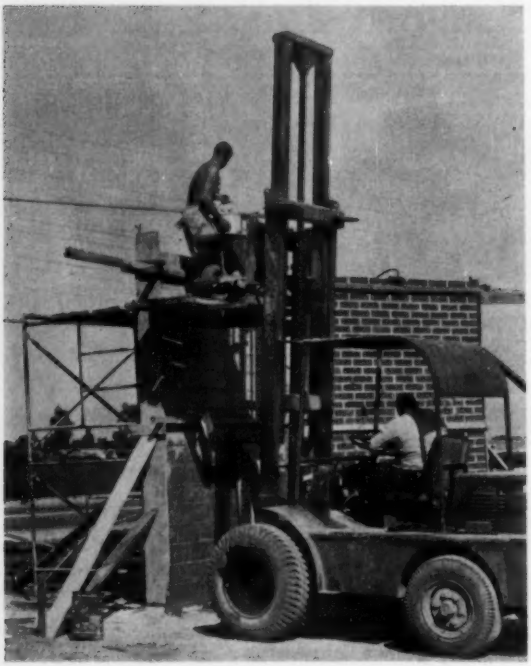

Fig. 4—Grouting panels M and N

Panels G to J, inclusive, as noted in Table 1 were then constructed pilot size (6 x 4 ft) to check the effect of various methods of consolidation. Concrete consolidation and the bottom joint were greatly improved. Voids, sags, and pockets were at a minimum. Panel J (Fig. 5) was nearly perfect.

The grout was consolidated by tamping with a 1 x 2-in. wood pole, by an internal vibrator having a ⅞-in. head, and by vibrating the steel, all as noted in Table 1. Reconsolidation was by the same methods after each batch had had time for some of the excess water to escape or be absorbed, and for the grout to stiffen to reduce hydrostatic pressure on the forms.

It was then decided to recheck the methods of consolidation and to stress reconsolidation under full scale conditions, i.e., in 12 ft high panels. Panels L to N were 12 x 8 ft and Panels K-1 and K-2 were made pilot size to test the difference in effect between internal vibration and tamping. Cutting these panels to see what had taken place in the grout core continued to show that improved technique was fundamental. The

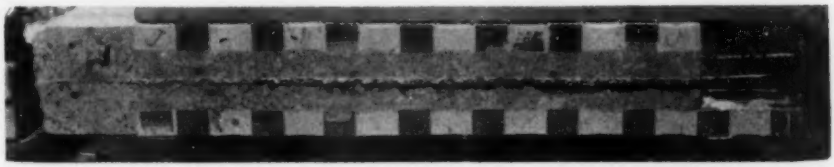

Fig. 5—Cut section of Panel J showing very good consolidation

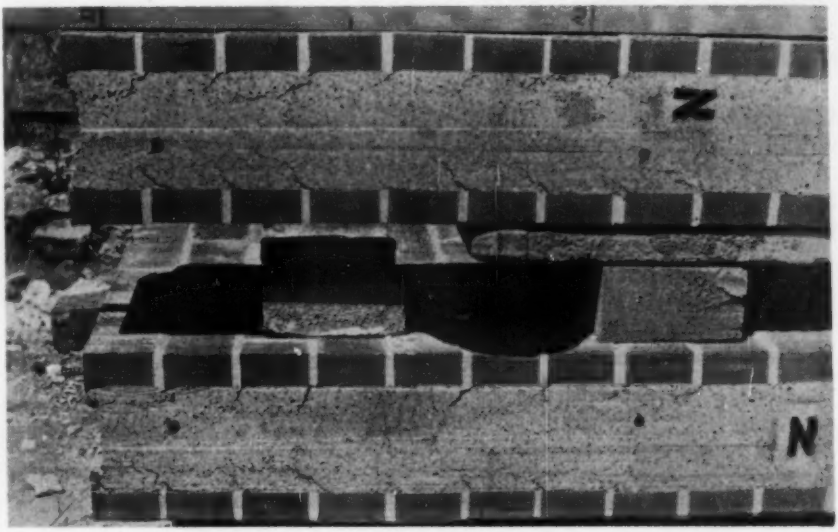

Fig. 6—Ineffective consolidation in top 2 ft of Panel N

Fig. 7—Effective but incomplete consolidation in middle portion of Panel N

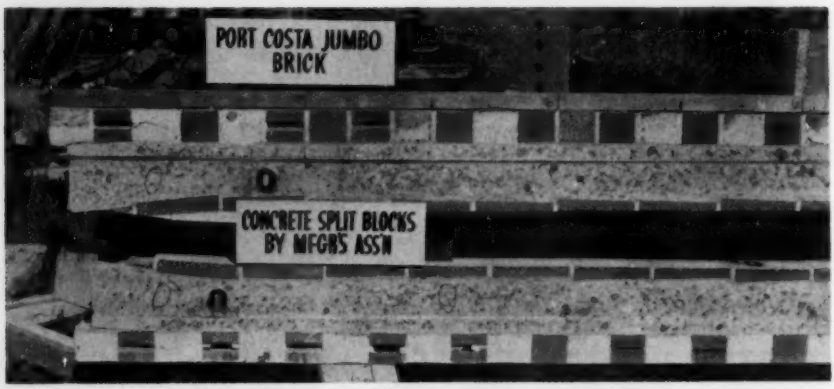

Fig. 8—Nearly perfect results in upper part of Panel O

Fig. 9—Good results in lower part of Panel O. Flaws were made purposely

Fig. 10—Middle section of Panel P shows effect of vibrating steel mat. Consolidation of grout around steel was good but contact of grout concrete with brick was uncertain

top 2 ft of Panel N (Fig. 6) showed a very distinct lack of reconsolidation as compared with the lower section of N (Fig. 7), even though the consolidation was accomplished by vibrating the steel mat. Laboratory notes were checked and revealed that the upper 2 ft had been hurried. This was an important clue. Full-sized panels O and P were then built to retest the reconsolidation technique. The grout in Panel O was tamped and retamped and in Panel P the steel mat was vibrated at intermittent periods.

Panel O was near perfect in consolidation of grout and contact with the brick as shown in Fig. 8 and 9. Panel P (Fig. 10) shows that consolidation is improved near the steel but vibrating the steel mat to affect the entire grout core was inconclusive.

Cores taken from reinforced grouted brick masonry, tested both in compression and in shear in the plane of contact between grout and the brick proved that compact and solidified cores give the best results. Cores having voids, airpockets, or laminations are not dependable, and are generally low in strength. Visual inspection is a rapid means of predetermining the dependability of the core to test out satisfactorily. In the test panel work the visual inspection method was used to decide whether the consolidation was adequate.

CONCLUSIONS

Effective reconsolidation produced the most solid grout core in Panels J and O. For many years concrete in nonabsorptive forms has been revibrated with beneficial results. This is clearly covered in an article, by Cornelius R. Barrett,[1] in which he answers the question, "How much time is available for revibration?" This article is referred to by Raymond J. Schutz[2] and his Fig. 7 shows the optimum period of re-

vibration of plain concrete compared to that with concrete having retarders added.

Using high slump concrete in *absorptive forms,* such as those of masonry, one must consider many variable factors in determining the optimum period of revibration. The absorptiveness of the masonry material must be considered, as well as the width of the grout space. The wider the space the more water must escape before the reconsolidation is effective. Weather conditions also play an important part in the speed of water loss from the grout.

Panels O and P tend to indicate that reconsolidation was performed in the optimum period for the masonry materials, thickness used, and prevailing weather conditions. Further tests are needed to determine the time factor to be applied for other form material available and for various thicknesses of grout. The minimum over-all thickness of walls are limited by code requirements. Varying weather conditions will also vary the time factor to apply.

As a result of the things learned from this project, guide specifications[4] for high lift grouting of clay brick masonry have been revised and the reconsolidation technique has been incorporated.

ACKNOWLEDGMENTS

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Title No. 57-34

Ultimate Strength of Reinforced Concrete Arches

By O. P. JAIN

As the load on an arch increases gradually, the portion of the rib near the section of maximum moment starts showing plastic deformations which result in an increase of horizontal thrust in the arch above that given by the elastic theory. This causes a reduction of maximum positive moment and an increase of maximum negative moment and thus the maximum moment in the arch is reduced resulting in a greater load bearing capacity of the arch. It is found that the actual ultimate strength of two-hinged arches of uniform section for various patterns of loading is 50 to 100 percent greater that the ultimate strength given by elastic theory. A method of calculation is presented which takes into account these plastic deformations. Complete agreement is found between the theoretical and test results. Although the calculations refer only to a single concentrated load on the arch, the same approach can be applied to find the ultimate strength for other loading patterns (including dead load).

■ IN PRACTICE, CONCRETE ARCHES are either two hinged or hingeless, three-hinged arches are seldom used. The following discussion is limited mainly to two-hinged arches, though the principles evolved can be used to advantage in the treatment of hingeless arches also.

A two-hinged arch is a statically indeterminate structure and the calculation of its end reactions requires knowledge of the deformation properties of the arch rib when subjected to thrust and bending moments. In such an analysis, it is usual to apply the elastic theory in a simplified manner, i.e., effect of shear deformations is neglected. Although the application of elastic theory to reinforced concrete redundant structures is not justified on the grounds that the concrete cracks in its tension zone and thus causes an alteration in the moment of inertia of the section as well as in the axis of the structure and that its modulus of elasticity changes with stress, yet the results obtained are not very inaccurate so long as the loads are within the working range. For practical purposes, therefore, the use of elastic theory is justified, especially when there is no alternative method which is more accurate.

But as loads increase, elastic theory gives appreciable error as the behavior of the materials deviate substantially from being elastic. Hence the pattern of bending moments in the arch changes with in-

creasing loads and the maximum moment in the arch near failure has quite a different relationship to the loads than in the working range. For example, if the relation between maximum moment M and load W in the working range is, $M = KW$, and if M_r is the maximum moment of resistance of the section at failure, then the failure load will appear to be equal to M_r/K. However, the actual failure load will be different since the factor K changes as failure approaches. Even the value of M_r cannot be estimated correctly with the use of existing methods based on straight line theory. Usual methods of analysis and design of reinforced concrete arches cannot, therefore, give the ultimate strength of the structure and the factor of safety adopted remains merely a fictitious number.

It is the purpose of this article to present a method of calculating the actual ultimate strength of two-hinged arches. The theoretical results have been verified by experiment and a complete agreement is found to exist.

Notation

M_r = maximum moment of resistance of a section at failure
H = horizontal thrust in the arch
E_c = Young's modulus of elasticity of concrete
E_s = Young's modulus of elasticity of steel
μ = bending moment at any point of the arch, treating it as a simply supported curved beam
ϕ = angular deformation per unit length of a member
f_c' = cylinder strength of concrete
d = effective depth of the section
Nd = distance of neutral axis of a section from its compression face
e_u = ultimate strain in concrete
e_c = strain in concrete at any stage
e_{cy} = strain in concrete at its elastic limit
e_s = strain in tensile steel at any stage
e = eccentricity of thrust at any section
f_s = stress in tensile steel
f_s' = stress in compressive steel

ANALYSIS ON ELASTIC THEORY

Let us consider circular arches with uniform rib section and having hinged supports at the ends. To measure experimentally the horizontal thrust in the arch at different stages of loading, the two ends were supported on rollers and were joined by a steel tie rod through hinged connections. The rod was calibrated beforehand, i.e., electrical resistance strain gages were fixed to it and a relation was obtained between the pull in the tie and the corresponding measured strain. Since the horizontal thrust in this arch is the only redundant force, its direct measurement affords a valuable means of checking the theoretical results.

The theoretical expression for the horizontal thrust H in the arch will be obtained by finding the horizontal movement of one end of the arch relative to the other and equating it to the stretch of the tie rod. In determining the horizontal movement of the ends of the arch, the effect of shear strains, being very small, will be neglected as is usually

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done. The effect of arch shortening will, however, be taken into account. Since the arch shortening effect is small compared with the flexural movements, it is justifiable to assume the normal thrust acting at each section of the arch equal to H for purposes of calculating the arch shortening, as it will simplify the calculations without appreciable errors. If A_1 is the sectional area of arch rib, I its moment of inertia, y the rise of arch at any point above the springings, A_2 the sectional area of tie rod, E_c and E_s the modulus of elasticity of concrete and steel, respectively, then we have horizontal movement of ends of arch

$$\Delta = \int (\mu - Hy)y \frac{ds}{E_c I} - \frac{HL}{A_1 E_c}$$

where μ is the moment at any point of the arch treating it as a simply supported curved beam, L is the span of the arch and ds is a small length along the arch axis.

$$\text{Stretch of tie rod} = \frac{HL}{A_2 E_s}$$

$$\therefore \int (\mu - Hy)y \frac{ds}{E_c I} - \frac{HL}{A_1 E_c} = \frac{HL}{A_2 E_s}$$

or

$$H = \frac{\int \mu y ds}{\int y^2 ds + L\left(\frac{I}{A_1} + \frac{E_c}{E_s} \cdot \frac{I}{A_2}\right)} \quad \ldots\ldots (1)$$

In the arches tested, span of arch $L = 120$ in., rise of arch $= 24$ in. The section of arch rib was 4 x 4 in. and the tie rod was a pipe of 0.59 sq in. sectional area. If the ratio E_s/E_c is taken as 15, we get

$$H = \frac{\int \mu y ds}{40{,}106 + 448}$$

If a point load W is applied at only the quarter point of this arch, then,

$$\int \mu y ds = 27{,}336\, W$$

$$\therefore H = \frac{27{,}336\, W}{40{,}106 + 448} = 0.674\, W$$

Knowing the value of H, it is now possible to draw the bending moment diagram for the arch. Vertical ordinates between the arch axis $A\,C\,B$ and the line of thrust $A\,O\,B$ in Fig. 1 give the moments in the arch under this condition of loading. The maximum sagging moment occurs at the load point E and maximum hogging moment occurs at a point F having coordinates (29.7, 18.56) with respect to end B of the arch. The

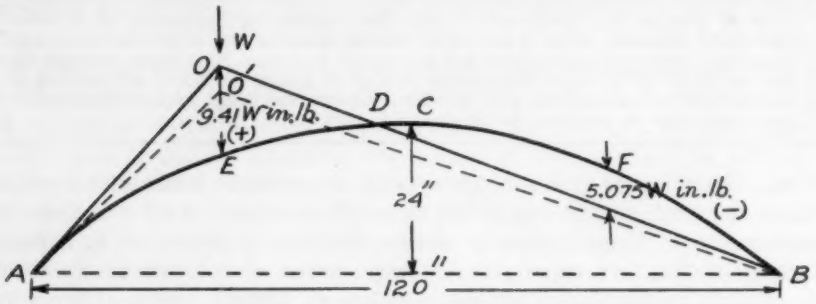

Fig. 1—Moments in arch under point load at quarter point

pattern of moments in the arch is such that the horizontal outward movement of springings due to positive (sagging) moments acting in the zone *AED* is balanced by the inward movement of springings due to negative (hogging) moments acting in the portion *BFD*. The normal thrust in the arch at the points of maximum positive and negative moment both is equal to 0.718 *W* lb.

The moment at the load point *E* being the largest, the strength of the arch will be governed by the moment of resistance of this section. The eccentricity of thrust at this section

$$e = \frac{9.41\,W}{0.718\,W} = 13.1 \text{ in.}$$

The vertical deflection of the load point works out to be $3110\,W/E_cI$ in.

The cube strength of concrete used in the test specimens was 3200 psi and the elastic limit of tensile steel was 45,000 psi. The modular ratio m may be taken as 15. Fig. 2 shows the section of the arch rib. For a thrust of 0.718*W* and bending moment of 9.41*W* in.-lb, the maximum compressive stress in concrete in this section works out to be $1.115\,W$ psi and tensile stress in steel comes to be $28.52\,W$ psi. With the maximum stress in concrete equal to 3200 psi and that in steel 45,000 psi, the value of load *W* will be governed by tensile stresses in steel and we get *W* from the equation, $28.52W = 45{,}000$, or $W = 1580$ lb. Thus, by elastic theory, the ultimate strength of the arch works out to be 1580 lb when loaded at one of the quarter points.

BEHAVIOR OF THE ARCH NEAR COLLAPSE

The elastic theory calculations assume that the materials obey Hook's law at all stages of loading and that the moment-deformation relation of the sections remains a straight line. However, it is well known that the angular deformation increases more rapidly than the moment as the ultimate moment of resistance of the section is approached. Fig. 3 shows a typical relation between moment *M* and the unit angular deformation ϕ of a reinforced concrete section. The section behaves

more or less elastically up to a bending moment M'. It will thus appear that the arch rib near point E, where the moments are more than M' (Fig. 3), will show extra angular deformations (plastic) over and above the elastic deformations. If $\Delta\phi$ is this extra angular deformation per unit length of the arch rib at any point, then the ends of the arch will show extra outward movement equal to $\Sigma\Delta\phi ds y$, the summation being for that portion of the arch which develops plastic strains. But since the springings of the arch are held against any outward movement,* due to plastic deformations. The total horizontal thrust H is then

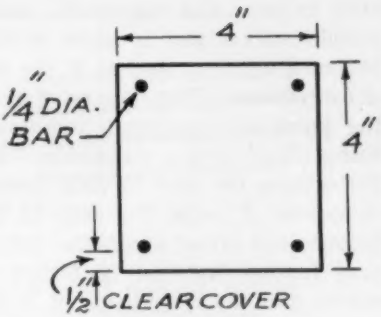

Fig. 2—Section of arch rib

$$H = \frac{\int \mu\, y ds + E_c I \Sigma\, \Delta\phi\, y ds}{\int y^2 ds + L\left(\frac{I}{A_1} + \frac{E_c}{E_s} \cdot \frac{I}{A_s}\right)} \qquad (2)$$

Eq. (2) shows that as the load approaches the ultimate strength, the horizontal thrust H in the arch increases over that given by elastic theory. This results in the reduction of positive moment near E and an increase in the negative moment near F. The line of thrust moves downward as shown by the dotted line $A\,O'$B in Fig. 1. The plastic strains at E thus result in a tendency to equalize the maximum positive and negative moments in the arch. This is called moment redistribution. The extent of this equalization depends upon the magnitude of plastic angular deformation which the sections near E can undergo before the breakdown of materials takes place. The portion of the arch rib which undergoes these plastic deformations is called the plastic hinge. As the moment at E decreases, the load bearing capacity of the arch increases. The most favorable value of H is that for which the positive and negative moments become equal, as in such a case, the maximum moment in the arch will be the least and the arch will be

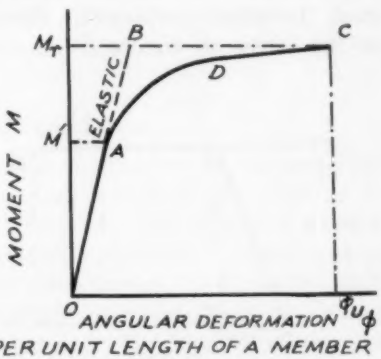

Fig. 3—Relation between moment and angular deformation

*The supports will have to exert extra inward horizontal reaction on the arch to counter balance its outward movement.

able to bear the maximum load. This value of H can be found by simple statics and is equal to 0.805 W. But before the moment at F becomes equal to that at E, the sections near F will also exhibit plastic deformations. This means that a second plastic hinge is formed at the point of maximum negative moment. The structure thus transforms itself into a mechanism with four hinges and shows abnormal deflections for any further increase in load W. The plastic deformations near F cause the ends of the arch to move inward and counterbalance the effect of plastic deformations near E. This results in stopping further increase of thrust H i.e., the quantity H/W, to a great extent and the magnitude of H seldom reaches the value 0.805 W. The actual value of H lies between 0.674 W and 0.805 W. The arch will collapse at this stage showing large deflections and crushing of concrete at E and F.

CALCULATION OF ULTIMATE STRENGTH

As concrete and steel both show plastic strains near collapse of the arch, the ordinary straight line theory cannot be applied to find the stresses and strains in sections approaching failure. This condition occurs only in small zones near E and near F. The rest of the arch rib still behaves more or less elastically. To calculate stresses and strains in sections showing yielding, the author has developed a plastic theory[1] which can be used with advantage in all such cases. Briefly, the theory assumes an idealized stress-strain curve of concrete as shown in Fig. 4. The maximum stress developed is assumed to be 85 percent of the cylinder strength f_c' of concrete and the section is assumed to fail only after the concrete has attained an ultimate strain e_u of 0.3 percent where there is no lateral restraint on concrete and 0.6 percent or more in case of lateral restraint. The slope of line $O\,A$ gives the effective modulus of elasticity of concrete which may be taken equal to about 1400 times the cylinder strength of concrete. For steel, its actual stress-strain curve is taken into consideration. So long as the stress in concrete remains less than 0.85 f_c' and that in steel less than its elastic limit, ordinary straight line theory is applicable. For higher strains, the calculations are based on the curve in Fig. 4. The stresses and strains are calculated by using equations of statical equilibrium and compatibility of strains. The strain in concrete is assumed to vary linearly across the section.

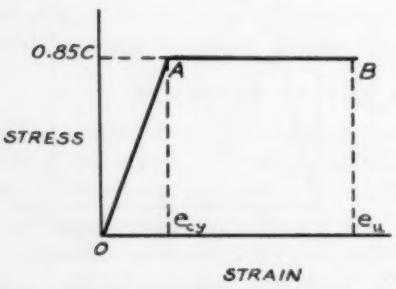

Fig. 4—Stress-strain curve of concrete

To calculate the ultimate strength of the arch, first a suitable value

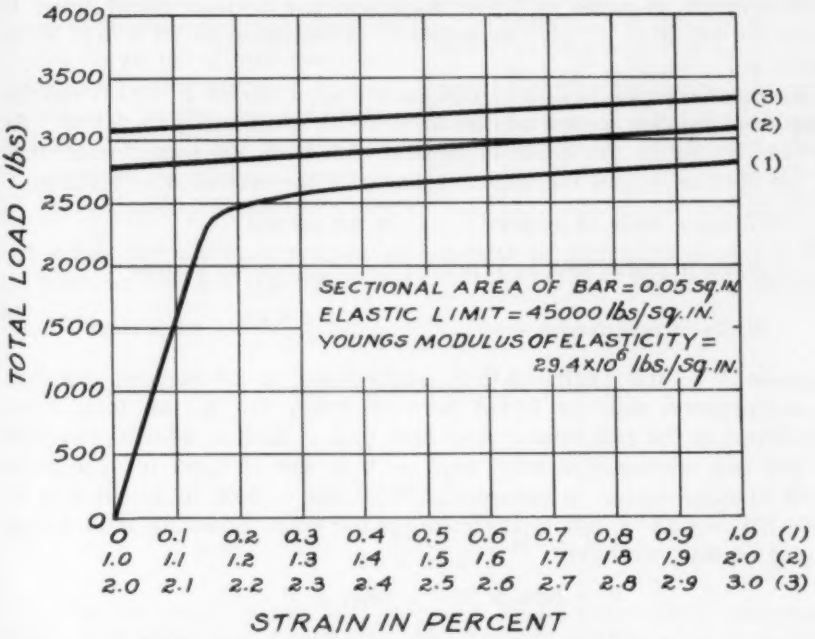

Fig. 5—Load-strain curve of 1/4-in. diameter mild steel bar

of H is assumed lying between the elastic value of 0.674 W and the fully plastic value of 0.805 W. After doing this, the moments and thrusts in the entire arch rib are calculated in terms of W. Then knowing the details of sections of the arch rib and its ultimate moment of resistance, the value of W can be found by equating the maximum bending moment to the ultimate moment of resistance. Knowing W, the quantity $\Sigma\Delta\phi yds$ required in Eq. (2) can be calculated and then the value of H determined from Eq. (2). For the calculated value of W to be correct, the assumed value of H should be equal to the calculated value of H.

METHOD ILLUSTRATED

Fig. 1 and 2 show the axis and the section of the arch rib, respectively. Here, cylinder strength, f_c', of concrete used was 2760 psi, (0.85 f_c' = 2350 psi), modulus of elasticity $E_c = 3.84 \times 10^6$ psi, modulus of elasticity of steel $E_s = 29.4 \times 10^6$ psi, modular ratio $m = 7.65$, area of steel in tension A_t = area of steel in compression $A_c = 0.1$ sq in. Ultimate strain of concrete $e_u = 0.6$ percent and its yield strain e_{cy} (corresponding to point A in Fig. 4) is 0.0612 percent. The load W was spread in a 4 in. length of rib at E. Let us assume $H = 0.77\,W$. Then the moment at the load point E, having coordinates (30, 18.6) with respect

to support A, is equal to $7.7\,W$ in.-lb and the normal thrust there is equal to $0.81\,W = P$, say: eccentricity of thrust, $e = 7.7\,W/0.81\,W = 9.5$ in.

We have now to find the ultimate value of thrust P which can be supported by the section of the arch at an eccentricity of 9.5 in. At failure, let Nd be the depth of neutral axis from the compression face of the section, where the effective depth of the section $d = 3.375$ in.

$$\text{Ultimate strain of concrete} = 0.6 \text{ percent}$$

$$\text{Strain in tensile steel } e_s \text{, at failure} = \frac{1 - N}{N} \times 0.6 \text{ percent}$$

$$\text{Strain in compressive steel} = \frac{Nd - 0.625}{Nd} \times 0.6 \text{ percent}$$

Suppose $N = 0.2$. Then strain in tensile steel is 2.4 percent and that in compression steel = 0.0444 percent. From Fig. 5, the total force developed in the two tensile steel bars $= 2 \times 3200 = 6400$ lb and that in the two compression steel bars $= 2 \times 650 = 1300$ lb. The total force of compression in concrete $= 3.23 f_c' Nd = 6000$ lb and it acts at $0.474\,Nd = 0.32$ in. below the compression edge. Equating total forces on the section, we have

$$P_1 + 6400 = 6000 + 1300, \text{ if } P = P_1$$

$$P_1 = 900 \text{ lb}$$

Taking moment of all forces about the center of tensile steel, we have

$$P_2\,(11.5 - 0.625) = 6000\,(3.375 - 0.32) + 1300 \times 2.75, \text{ if } P = P_2$$

$$P_2 = 2010 \text{ lb}$$

The correct value of N is that for which $P_1 = P_2 = P$. Table 1 gives the values of P_1 and P_2 for various values of N.

It will be seen that P_1 becomes more than P_2 as N changes from 0.2 to 0.22. Hence the correct value of N lies between 0.2 and 0.22, and can be obtained by finding the point of intersection of two curves plotted between (N, P_1) and (N, P_2). The correct value of $N = 0.21$ and the thrust $P = 2380$ lb. The angular rotation per unit length of rib at this point $= 0.006/Nd = 0.006/(0.21 \times 3.375) = 8.45 \times 10^{-3}$ radians
Ultimate load W on the arch $= P/0.81 = 2380/0.81 = 2940$ lb

$$\therefore H = 0.77\ W = 0.77 \times 2940 = 2260 \text{ lb}$$

Having found the values of W and H, it is now possible to calculate moments and thrusts at all points. Then the quantity $\Sigma \Delta\phi y ds$ can be calculated. Here $\Delta\phi$ is the difference between the total angular rotation at a point (including effect of plastic deformations) and the rotation if the materials had remained elastic up to that stage. The zones of maximum positive moment as well as maximum negative moment both will be investigated to calculate the quantity $\Sigma \Delta\phi y ds$.

For this purpose, points will be taken some distance apart on either side of the points of maximum moment, until the sections do not show plastic deformations. At these sections, the concrete strains will be less than the ultimate strain of 0.6 percent and will be unknown.

TABLE I—VALUES OF THRUST FOR VARIOUS VALUES OF N

N	0.20	0.22
Force in tension steel	6400	5600
Force in compression steel	1300	2740
Force in concrete	6000	6600
P_1	900	3740
P_2	2010	2520

Let e_c be the maximum strain in concrete at any section and ϕ be the angular rotation per unit length at that section. Referring to Fig. 6,

$$e_{cy} = \frac{0.85 f_c'}{E_c} = \frac{0.85 f_c'}{3.84 \times 10^6}$$

Also,

$$\phi = \frac{e_{cy}}{z}$$

$$\therefore z = \frac{e_{cy}}{\phi} = \frac{0.85 f_c'}{3.84 \times 10^6 \phi} = \frac{612 \times 10^{-6}}{\phi}$$

Let P be the thrust at any section and e' be its eccentricity measured from the tensile steel. Let Nd be the depth of neutral axis, f_s be the stress in tensile steel, and f_s' be the stress in compression steel. Now equating forces on the section, we have

$$P + A_t f_s = (Nd - z)b \times 0.85 f_c' + \tfrac{1}{2}bz \times 0.85 f_c' + A_c f_s'$$

where A_t and A_c are the areas of tensile and compression steel in the section, and b is the width of section.

or,

$$P + 0.1 f_s = 9400 \left[3.375 N - \frac{306 \times 10^{-6}}{\phi} \right] + 0.1 f_s' \quad \text{.....(3)}$$

Taking moment of all forces about the tensile steel, we have

$$Pe' = (Nd - z)b \times 0.85 f_c' \left[d - \frac{Nd - z}{2} \right]$$

$$+ \tfrac{1}{2}bz \times 0.85 f_c' \left[d - Nd + \tfrac{2}{3}z \right] + A_c f_s' \times 2.75$$

or,

$$Pe' = 4700 \left[22.78 N - 11.39 N^2 + \frac{2065 \times 10^{-6} N}{\phi} - \frac{2065 \times 10^{-6}}{\phi} \right.$$

$$\left. - \frac{0.125 \times 10^{-6}}{\phi^2} \right] + 0.275 f_s' \quad \text{.....(4)}$$

The effective value of f_s' is equal to

$$(Nd - 0.625)\, \phi\, (m - 1)\, E_c = 86.1 \times 10^6 \phi\, (N - 0.185) \quad \text{.....(5)}$$

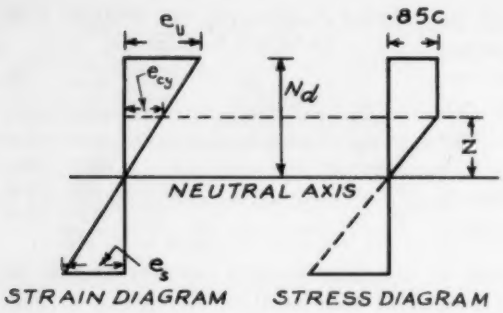

Fig. 6 — Stress-strain relations in flexure in a reinforced concrete section

when the compression steel and concrete at its level are both elastic.

$$[(Nd - 0.625)\,\phi\, m\, E_c - 0.85\, f_c'] = 99 \times 10^6 \phi\,(N - 0.185) - 2350 \quad \text{.....(6)}$$

when compression steel is elastic, while the concrete at its level has become plastic.

$$(f - 0.85\, f_c') = (f - 2350) \quad \text{.....(7)}$$

when compression steel and concrete at its level are both plastic and f is the actual stress in compression steel. The appropriate value of f_s' should be used in Eq. (3) and (4). For known values of P and e', the unknowns are N and ϕ in these equations and their values can only be found by trial and error. First a value of ϕ is assumed and value of N is calculated from Eq. (4). Then substituting the values of ϕ and N in Eq. (3), f_s is found. Also the strain e_s in tensile steel is equal to $(1 - N)d\phi$ which can now be worked out. For the assumed value of ϕ to be correct, the value of f_s and e_s as calculated above, should satisfy the stress-strain curve of steel. To illustrate the working of above equations, let us analyze the section of maximum negative moment which occurs at the point (33.1, 19.75) with respect to the end B as origin in Fig. 1. If θ is the angle which the tangent at this point makes with the horizontal, $\sin\theta = 0.309$, $\cos\theta = 0.95$, $M = 20{,}300$ in.-lb; $P = 2377$ lb, $e' = 9.93$ in.

Let ϕ be assumed to be equal to 2×10^{-3} radians and let the compression steel and concrete at its level be both in the elastic stage. Then Eq. (4) gives

$$2377 \times 9.93 = 4700\,[22.78\,N - 11.39\,N^2 +$$

$$\frac{2065 \times 10^{-6} N}{2 \times 10^{-3}} - \frac{2065 \times 10^{-6}}{2 \times 10^{-3}} - \frac{0.125 \times 10^{-6}}{4 \times 10^{-6}}\Big]$$

$$+ 0.275 \times 86.1 \times 10^{-6} \times 2 \times 10^{-3}\,(N - 0.185)$$

$$\therefore\ N = 0.244$$

Substituting these values of N and ϕ in Eq. (3), we get

$$2377 + 0.1 f_s = 9400\left[3.375 \times 0.244 - \frac{306 \times 10^{-6}}{2 \times 10^{-6}}\right] + 0.1 \times 86.1 \times 10^6 \times 2 \times 10^{-3} \times (0.244 - 0.185)$$

or,

$$0.1 f_s = 5400$$

$$e_s = (1 - N) d\phi = (1 - 0.244)\, 3.375 \times 2 \times 10^{-3} = 0.51 \text{ percent}$$

These values of f_s and e_s satisfy the stress-strain curve of steel shown in Fig. 5. Hence the analysis is correct and the angular deformation at this section is 2×10^{-3} radians.

Now the angular deformation will be calculated at the same section on the assumption that materials remain elastic. On the usual elastic

TABLE 2 — CALCULATION OF $\Delta\phi$ IN POSITIVE MOMENT ZONE

Distance of section from load point, in.	y	ϕ	ϕ_E	$\Delta\phi$
6	16.4	0.66×10^{-3}	0.645×10^{-3}	0.015×10^{-3}
4	17.19	1.1×10^{-3}	0.745×10^{-3}	0.355×10^{-3}
2	17.94	4.5×10^{-3}	0.835×10^{-3}	3.665×10^{-3}
0	18.66	8.45×10^{-3}	0.895×10^{-3}	7.555×10^{-3}
2	19.36	6.3×10^{-3}	0.855×10^{-3}	5.445×10^{-3}
4	20.00	1.25×10^{-3}	0.734×10^{-3}	0.516×10^{-3}
6	20.6	0.62×10^{-3}	0.62×10^{-3}	0

TABLE 3 — CALCULATION OF $\Delta\phi$ IN NEGATIVE MOMENT ZONE

Distance of section from point of maximum moment, in.	y	ϕ	ϕ_E	$\Delta\phi$
14	14.4	0.66×10^{-3}	0.66×10^{-3}	0
12	15.2	0.74×10^{-3}	0.688×10^{-3}	0.052×10^{-3}
10	16.1	0.9×10^{-3}	0.706×10^{-3}	0.194×10^{-3}
8	16.9	1.12×10^{-3}	0.735×10^{-3}	0.385×10^{-3}
6	17.65	1.34×10^{-3}	0.75×10^{-3}	0.59×10^{-3}
4	18.4	1.70×10^{-3}	0.765×10^{-3}	0.935×10^{-3}
2	19.1	1.80×10^{-3}	0.77×10^{-3}	1.03×10^{-3}
0	19.75	2.0×10^{-3}	0.775×10^{-3}	1.225×10^{-3}
2	20.45	1.8×10^{-3}	0.770×10^{-3}	1.03×10^{-3}
4	20.95	1.7×10^{-3}	0.765×10^{-3}	0.935×10^{-3}
6	21.45	1.34×10^{-3}	0.75×10^{-3}	0.59×10^{-3}
8	21.9	1.12×10^{-3}	0.735×10^{-3}	0.385×10^{-3}
10	22.25	0.9×10^{-3}	0.706×10^{-3}	0.194×10^{-3}
12	22.6	0.74×10^{-3}	0.688×10^{-3}	0.052×10^{-3}
14	22.95	0.66×10^{-3}	0.66×10^{-3}	0

theory calculations, N will be found to be 0.31 and $\phi_E = 0.775 \times 10^{-3}$ radians, where ϕ_E is the elastic angular rotation.

$$\therefore \text{Net plastic deformation } \Delta\phi \text{ at this section} = (2 \times 10^{-3} - 0.775 \times 10^{-3}) = 1.25 \times 10^{-3} \text{ radians.}$$

Working as above, the values of $\Delta\phi$ will now be found at the various sections of the arch at an interval of 2 in. on either side of the points of maximum positive and negative moments. Tables 2 and 3 give the values of $\Delta\phi$ at these sections. The quantity $\Sigma\Delta\phi yds$ works out to be 656.7×10^{-3} units for the positive moment zone.

The quantity $\Sigma\Delta\phi yds$ works out to be 288×10^{-3} units for the negative moment zone. Therefore, the net outward movement of arch springings due to plasticity $= \Sigma\Delta\phi yds = (656.7 - 288)\ 10^{-3} = 368.7 \times 10^{-3}$ in. The assumed value of H (equal to 0.77 W) is admissible and is on the safer side if Eq. (2) gives H equal to or more than this assumed value. From Eq. (2), H is given by

$$H = \frac{27{,}336\,W + E_cI\Sigma\Delta\phi yds}{40{,}106 + 448}$$

$$= \frac{27{,}336 \times 2940 + E_cI \times 368.7 \times 10^{-3}}{40{,}554}$$

The quantity E_cI in the above equation should be a representative value for the entire arch rib. Its correct value is difficult to estimate due to the fact that some portions of the rib are uncracked where the value E_cI will be high and in other portions, the rib will crack in the tensile zone of the sections and there the value of E_cI will decrease considerably. If a higher value of E_cI is used, it will result in giving a higher value of H and the arch will appear to be stronger. On the other hand, a lower value of E_cI will tend to underestimate the strength of the arch, which again is undesirable. Hence it is necessary that a correct estimate be made for the quantity E_cI. The author has dealt with this problem in detail elsewhere[2] and it is recommended that the effective value of I be taken equal to the lowest value for the cracked section plus 1/3 *rd* the difference between this lowest value and the highest value for the uncracked section. In this case, the lowest value of N for the cracked section is 0.1 and so the lowest I works out to be 5.9 in.^{4} For uncracked sections, $I = 23.8$ in.^{4} Therefore, effective I for the entire arch $= [5.9 + 1/3\ (23.8 - 5.9)] = 11.9$ in.^{4}

$$\therefore E_cI = 3.84 \times 10^6 \times 11.9 = 45.7 \times 10^6 \text{ in.}$$

$$\therefore H = \frac{27{,}336 \times 2940 + 45.7 \times 10^6 \times 368.7 \times 10^{-3}}{40{,}554} = 2400 \text{ lb}$$

It will be seen that the assumed value of H is on the safe side. In fact, a higher value of H (greater than 0.77W) could be assumed and in that case, the ultimate load W will work out to be more. But it may

not be possible to further increase H substantially as this will result in higher negative moments in the arch and the net value of $\Sigma\Delta\phi yds$ will decrease very rapidly. It is therefore not proposed to repeat the calculations with a higher H/W ratio and thus the ultimate load W which the arch can substain is 2940 lb as against 1580 lb given by elastic theory. The percentage increase over elastic theory is 86 percent.

TEST RESULTS

The arch analyzed above was also tested to failure to verify the theoretical results. To get representative and reliable inferences, three specimens, similar in all respects, were prepared and tested. The load W was gradually increased and at every stage of loading, the following observations were recorded:

1. Load
2. Horizontal thrust H (i.e., tension in tie rod)
3. Vertical downward deflection of load point
4. Vertical upward deflection of the point of maximum negative moment
5. Strain measurements at five levels on the sections of maximum positive and negative moments

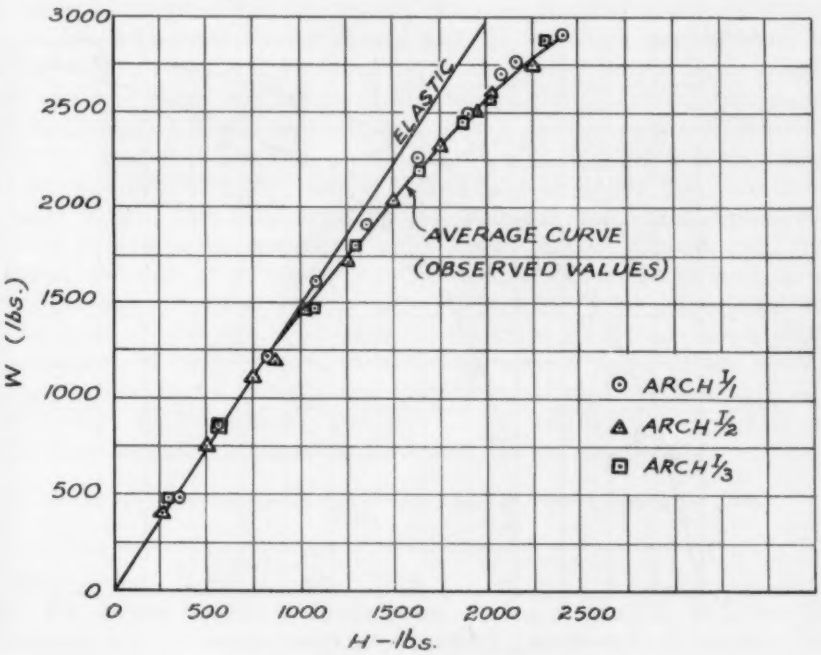

Fig. 7—W-H relation for Type I arch

TABLE 4—COMPARISON OF OBSERVED AND THEORETICAL VALUES

		Theoretical	Observed
W, lb		2940	2880
H, lb		2260	2440
H/W		0.77	0.778
Neutral axis coefficient N		0.21	0.208
Ultimate strain, percent	Concrete	0.6	0.56
	Tensile steel	2.26	2.13
E_cI		45.7×10^6	47.9×10^6

Fig. 7 and 8 show the curves giving a relation between W and H, and between W and vertical deflections of the two critical points E and F. From these curves, it is evident that the behavior of the arch is elastic in the initial stages of loading and then the effect of plastic deformation becomes more pronounced. From the load-deflection curve (Fig. 8), it can be seen that as failure approached, the deflections became abnormal and the arch behaved like a mechanism with hinges at four points—the two springings and the points of maximum positive and negative moments. There was no visible crushing of concrete in the arch even at the ultimate load and the failure was indicated by large

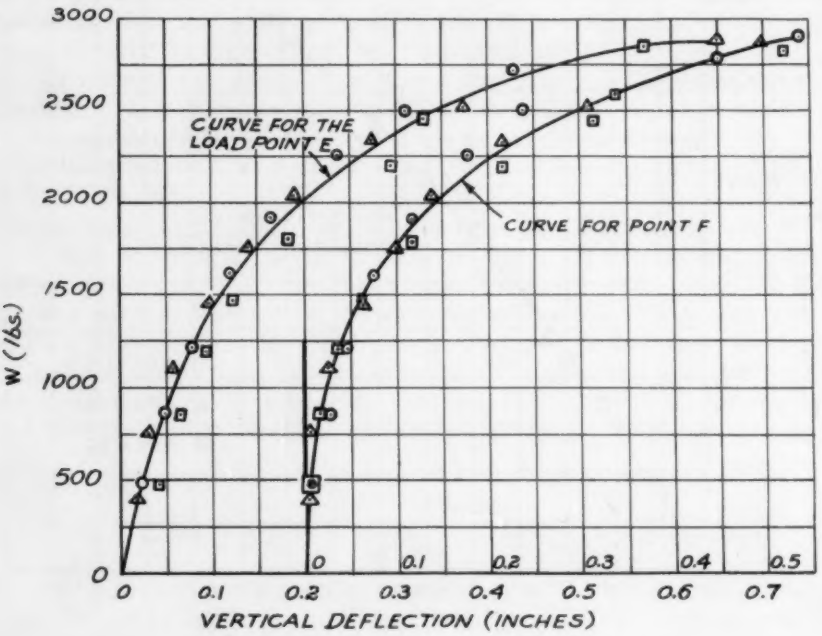

Fig. 8—Load-deflection curve for Type I arch

deflections and the inability of the jack to apply increased loads at the normal rate of pumping.

The effective value of E_cI can also be obtained from the load deflection curve in Fig. 8. The vertical deflection of the arch at the load point in the elastic stage works out to be $3110W/E_cI$. The observed deflection for a load of 1000 lb is 0.065 in.

TABLE 5—VALUES OF L_p FOR VARIOUS THRUST ECCENTRICITIES

Eccentricity of thrust in terms of depth D of section	L_p, percentage of span
$3D$	3.1
$2D$	2.2
D	1.9
$D/2$	1.7

$$\therefore E_cI = \frac{3110 \times 1000}{0.065} = 47.9 \times 10^6 \text{ units}$$

It will be seen that the estimated value of E_cI is reasonable and on the safe side. Table 4 shows the observed and theoretical values for some of the more important quantities. There is almost perfect agreement between theoretical and observed results.

THE PLASTIC HINGE

The most tedious part of the calculations is to find the value of $\Sigma\Delta\phi yds$, i.e., the inelastic rotation of the plastic hinge. This inelastic rotation will depend upon the length of the rib showing plastic deformations, the maximum strains caused and the depth of neutral axis. If this length of the rib is replaced by an imaginary length L_p in which the concrete fibers are supposed to undergo the ultimate strain e_u at all sections and in which the depth of the neutral axis also remains constant such that the total plastic rotation in this length is the same as actually occuring, then this length L_p may be called the "length of plastic hinge." The total rotation will then be equal to $L_p \times e_u/Nd$ where e_u is the ultimate strain of concrete. L_p will depend upon the rate of decrease of moment on either side of the point of maximum moment and it's value can be established, after some experience, as a function of span of the member for different conditions of variation of moment. If L_p is known, then the inelastic rotation of the plastic hinge can be found quickly and calculation of ultimate load will become easy. In the present case, $e_u = 0.006$, Nd is $(0.21 \times 3.375) = 0.71$ in. and total rotation of plastic hinge at E is

$$\Sigma \Delta \phi \, ds = \frac{656.7 \times 10^{-6}}{18.66} = 35.2 \times 10^{-3} \text{ radians}$$

$$\therefore L_p = \frac{35.2 \times 10^{-3} \times 0.71}{0.006} = 4.15 \text{ in.}$$

i.e., the length of plastic hinge is nearly equal to 1/30 of span. By working out the length of plastic hinge L_p in the above manner for various cases, some rules can be framed to get the value of L_p easily.

TABLE 6—COMPARATIVE TEST RESULTS ON ARCHES

Arch type	Eccentricity of thrust	Percentage of tested strength	
		Ultimate strength as given by elastic theory	Ultimate strength as given by proposed theory
1	3D	55.0	102
2	3D	59.8	92.5
3	3D	55.2	96.0
4	3D	57.7	94.0
5	3D	63.2	96.0
6	2D	67.2	89.0
7	2D	48.2	93.0
8	2D	46.3	87.0
9	D	50.0	85.0
10	D/2	55.5	85.0

This will reduce the labor involved in ultimate load calculations by trial and error.

The value of L_p was found to depend upon the eccentricity of thrust, i.e., the ratio of moment to thrust at the point of maximum moment. The author had occasion to determine the value of L_p for various values of eccentricity of thrust. The results are given in Table 5.

CONCLUSION

From the above, it is clear that elastic theory calculations cannot estimate the ultimate strength of arches even approximately. To get a true idea of the factor of safety, ultimate load theory calculations are necessary. The proposed method of calculation gives results which agree completely with observed values. The author tested arches under different conditions of loading such that the eccentricity of thrust at the section of maximum moment varied from $D/2$ to $3D$. The lower eccentricity of thrust corresponds almost to a condition of dead loading. The other variables tried were the quality of concrete and the percentage of reinforcement in the section. In all, ten sets of arches were tested having ten different conditions. Table 6 gives the comparative results for these arches. It will be seen that in each case, the elastic theory underestimated the ultimate strength by about 40 percent, while the proposed theory gives results which agree closely with the test observations. This method of calculation can be applied to all types of statically indeterminate structures.

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Title No. 57-35

Resistance to Shear of Reinforced Concrete Beams

Part 5 — Anchorage and Bond

By J. TAUB and A. M. NEVILLE

It is shown that hooks at the ends of plain round bars materially increase the load-carrying capacity of a beam failing in shear-tension. Means of lessening the destructive action of hooks are indicated. Beams with deformed bars and vertical stirrups show a considerable resistance to slip, so that composite action in the end parts of the beam is well preserved. This is not necessarily so when no stirrups are present as in some cases the wedging action of the deformations may tend to split the concrete. Bond failure is shown not to be a primary cause of failure but merely a consequence of the redistribution of internal forces following the widening of the diagonal tension crack. For this reason the value of the nominal bond stress at failure depends on the effectiveness of the shear reinforcement of the beam, and the use of a fixed permissible bond stress is shown to result in a greatly varying factor of safety.

■ IN THE DESCRIPTION OF SHEAR-TENSION failures in Parts 1, 2, and 3 of this paper* it was mentioned that the destruction of the beam end by the hooks can be the direct cause of the collapse of the beam. Since this shows a connection between "shear" and "anchorage" it is proposed now to discuss the effects of hooks in detail. This will be followed by a study of bond as related to shear.

Notation

a = shear span
d = effective depth of beam
f_c' = compressive strength of concrete test cylinder
f_{cu} = compressive strength of concrete test cube
jd = lever arm of internal forces in beam
kd = depth of compression zone of concrete
o = sum of perimeters of bars in tension reinforcement
P_c = total load on beam at formation of first diagonal tension crack
P_s = total load on beam at first slip of the end of tension steel
P_u = total load on beam at failure
p = percentage tension steel area
r = percentage web reinforcement area
u = nominal bond stress
u_c = nominal bond stress at development of first diagonal tension crack
u_u = nominal bond stress at failure
V = total shearing force
v = nominal shearing stress in concrete
v_c = nominal shearing stress at formation of first diagonal tension crack
v_s = nominal shearing stress at first slip of the end of tension steel
v_u = nominal shearing stress at failure

*Part 1 appeared in Aug. 1960 ACI JOURNAL, p. 193; Part 2, Sept. 1960, p. 315; Part 3, Oct. 1960, p. 443; Part 4, Nov. 1960, p. 517.

ROLE OF HOOKS IN THE SHEAR RESISTANCE OF BEAMS

A convenient starting point of this discussion is the consideration of Beam 1025, described in Table 15. The quantity of shear reinforcement in this beam was one half of that required by the standard elastic theory of the British Code of Practice.[34*] When subjected to a uniformly distributed load the behavior of this beam was similar to that of a beam with inadequate web reinforcement but failure occurred, nonetheless, by the yield of the main tension steel at midspan, under a load equal to the ultimate load of a beam with full shear protection. The question arises how Beam 1025 achieved such a high strength.

After the redistribution of internal forces and the subsequent destruction of the bond between the main steel and the concrete right up to the support, the tension force in this steel became constant between the crack and the support, and the mode of action of the beam changed into that of a two-hinged tied arch (see Part 1). Thus the load-carrying capacity of the beam came to depend on the efficiency of the end anchorage of the tension steel, i.e., on the hooks. This explains why in a beam in which *shear-tension failure has started* the hooks at the ends of the tension steel are a vital factor in the shear resistance of the beam. In beams with web reinforcement the role of hooks starts only when the action of bent-up bars and stirrups as shear reinforcement has come to an end, and the bond between the tension steel and the concrete has been destroyed right up to the support of the beam.

In Beam 1025 the cross-sectional area of the tension steel in the outer parts of the beam was very large, and for this reason the stress in this steel, and consequently the pressure of the hooks on the concrete, was diminished. In addition, since the beam was made of high quality concrete it could resist the stresses induced by the hooks.

The description of the mode of failure of Beam 1025 has shown the importance of the end hooks when the shear reinforcement of a beam is otherwise inadequate, and the beam is so proportioned that incipient shear-tension failure arises. This type of failure is less frequent in rectangular than in T-beams and its occurrence is governed also by the ratio of the shear span to the effective depth of the beam, a/d. All these factors were exhaustively discussed in Parts 1 and 2.

A more general picture of the role of the hooks on plain steel bars is obtained from a comparison of the behavior of beams with and without hooks, both with and without web reinforcement. Tables 27 and 28 give data on various T-beams,[18, 32] and it can be seen that the influence of the hooks is relatively greater the less sufficient the shear reinforcement provided; the same effect is apparent when bent-

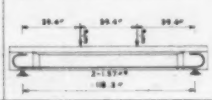
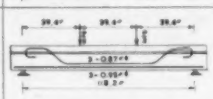
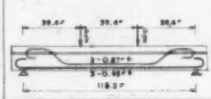
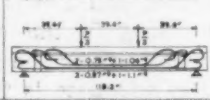
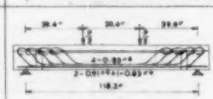
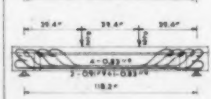
*References are numbered consecutively beginning with Part I of this five-part series.

up bars or vertical stirrups are used. This observation is in accordance with the earlier deductions: the more effective the shear reinforcement the smaller the opening of the diagonal tension cracks and the lower the stress in the main reinforcement transmitted to the hooks, and hence the lower the stresses exerted by the hooks on the surrounding concrete.

From a comparison of Beams 12 and 23 it can be further seen that semicircular hooks, as commonly used, are more effective than right-angled hooks. The increase in strength is of the order of 17 percent.

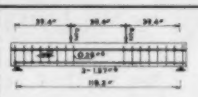
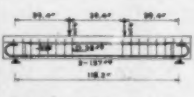
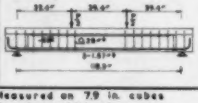
Some interesting experiments on the role of hooks were performed by Lewis[43] who tested beams both with mild steel and with Tentor reinforcement. The latter is a type of high strength deformed bar produced from mild steel round bars by simultaneous twisting and

TABLE 27—INFLUENCE OF HOOKS ON SHEAR STRENGTH OF T-BEAMS WITHOUT VERTICAL STIRRUPS

Investigator	Beam No.	Beam Details	a/d	f_{cu} (*) psi	p (**) percent	P_u kips	v_u psi	$\frac{v}{f_{cu}}$	Increase in P_u due to hooks percent	Mode of failure
Bach and Graf (28), (32)	1		2.9	3440	3.6	35.9	185	0.054		Slip of tension steel
	7		2.9	3440	3.6	54.5	286	0.083	52	Destruction of beam end by hooks
	28		2.9	3440	3.6	77.6	420	0.122		Slip and pressing down of tension steel
	29		2.9	3440	3.6	92.6	500	0.145	19	Pressing down of tension steel
	37		2.9	3440	3.6	98.5	511	0.149		Slip of straight steel and yield of remaining bars at mid-span
	38		2.9	3440	3.6	109.1	573	0.156	11	Yield of tension steel at mid-span
	41		2.9	3440	3.6	98.4	515	0.150		Slip of straight steel and yield of remaining bars at mid-span
	42		2.9	3440	3.6	104.3	546	0.159	6	Yield of tension steel at mid-span

Notes: (*) Measured on 7.9 in. cubes
(**) Based on width of stem
All steel consists of plain round bars

TABLE 28—INFLUENCE OF HOOKS ON SHEAR STRENGTH OF T-BEAMS WITH VERTICAL STIRRUPS

Investigator	Beam No.	BEAM DETAILS	a/d	f'_{cu}[a] psi	p[b] percent	r percent	P_u kips	v_u psi	$\frac{v_u}{f'_{cu}}$	Increase in P_u due to hooks	Mode of failure
Beck and Graf [30]	4		2.9	3440	3.6	0.26	48.5	255	0.074		Slip of tension steel
	12		2.9	3440	3.6	0.26	79.5	419	0.122	64	Destruction of beam end by hooks
	23		2.9	3440	3.6	0.26	67.7	356	0.103	40	Opening of hooks

Notes: (a) Measured on 7.9 in. cubes
(b) Based on width of stem
All steel consists of plain round bars

stretching. From Lewis'[43] test data, summarized in Table 29 it can be seen that in beams without web reinforcement the addition of hooks to mild steel reinforcement results in a greater relative increase in the load capacity of the beam than in the case of deformed bars. Furthermore, the hooks prevent the free pull-out of the mild steel bars.

In beams with stirrups the influence of the hooks is smaller still. In beams with deformed bars and stirrups, Lewis' tests show that the hooks result in a small increase in the load-carrying capacity of a beam, although their influence is smaller than when plain round bars are used. It may be noted that mechanical wedging between the bar deformations and the concrete becomes effective as soon as a small relative displacement has taken place.

Some further indication of the role of hooks in deformed bars can be obtained from the tests of Gaston, Siess, and Newmark.[24] It appears that a beam with deformed bars with hooks (T2Mb) failed in shear under a load 6 percent higher than a similar beam without hooks (T2Ma) even though the latter beam was made of concrete some 7 percent stronger than the beam with hooks.

No general conclusions can, however, be drawn and Lewis' data[43] cannot be used for quantitative comparisons owing to the variable concrete strength of different beams but they again confirm the general statement that hooks are more important the less adequate is the web reinforcement provided in the first instance.

Deformed bars are generally used without hooks and it would be reasonable to compare the behavior of beams reinforced by plain bars with hooks and by deformed bars without hooks. This was done by Holmgren[44] who used deformed bars with widely spaced corrugations (at twice the diameter of the bar). He found that plain bars with hooks exhibited a lower slip than deformed bars but the latter showed better crack patterns, i.e., more numerous fine cracks rather than

fewer wide ones as shown by the plain bars. On the other hand, Whitney[23] observed that deformed bars tend to split the concrete due to the wedging action of the deformations, particularly when these are high and when the steel is highly stressed.

This is in agreement with the authors' interpretation of some of the test results of Gaston, Siess, and Newmark.[24] A beam with deformed bars with hooks (T2Mb) failed in shear under a moment of 29.31 ft-kip while a similar beam with plain round bars with hooks (T1Mb) reached its full flexural strength of 32.83 ft-kip. The concrete strengths varied somewhat (4750 psi for T1Mb and 4020 for T2Mb) but the main reason for the difference is believed to be due to the accelerated split-

TABLE 29—INFLUENCE OF HOOKS ON SHEAR STRENGTH OF RECTANGULAR BEAMS

Investigator	Beam No.	BEAM DETAILS	Type of steel	$\frac{a}{d}$	f'_{cu} psi	p percent	r percent	P_u kips	v_u psi	$\frac{v_u}{f'_{cu}}$	Ratio of values of the quotient $\frac{v_u}{f'_{cu}}$ for beams with and without hooks
Lewis[43]	D1		mild	1.33	2570	0.97	0	22.4	158	0.061	1.3
	D2		mild	1.33	3790	0.97	0	42.3	299	0.079	
	D4		mild	1.33	5300	0.97	0.73	30.7	217	0.041	2.0
	D3		mild	1.33	4310	0.97	0.73	49.7	350	0.081	
	C1		tentor	1.32	2570	0.54	0	29.1	203	0.079	1.0
	C2		tentor	1.32	3790	0.54	0	41.7	291	0.077	
	C4		tentor	1.32	5300	0.54	0.73	56.2	392	0.074	1.3
	C3		tentor	1.32	4310	0.54	0.73	59.6	416	0.097	

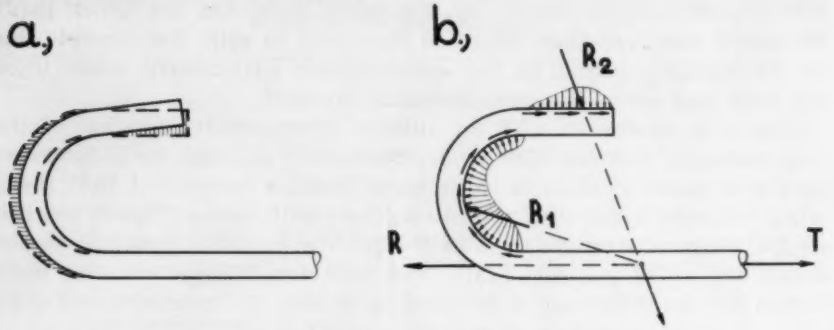


Fig. 35—(a) Deformation of a hook; (b) distribution of pressure

ting of the beam by the deformed bars *when no stirrups are present.* It is, therefore, believed that when deformed bars are used the provisions of stirrups is essential.

The action of welded anchorage plates can be expected to be similar to that of hooks. Some indication of this is obtained from Ferguson and Thompson's[9] tests on T-beams with deformed bars. For example, Beam C1 failed in shear-tension by the destruction of the beam end in a manner similar to the failure of Beam 7 (with hooks) of Table 27.

A comparison of two T-beams of Joistile type tested earlier by the same authors,[45] and differing only in the provision of a welded anchorage plate in Beam H-13 and its absence in Beam H3, shows that the nominal shearing stresses at failure of the two beams differed considerably. The actual values were 345 and 260 psi, respectively. Thus end anchorage on deformed bars appears to have increased the load carrying capacity of the beam by 33 percent.

THE ACTION OF HOOKS

The deformation of a hook and the distribution of pressure exerted by it on the concrete are represented in Fig. 35. Fig. 36 shows a clear picture of the movement of hooks in a T-beam at final collapse.

The pressure of the hook on the concrete produces in it tension in a transverse direction, and when this reaches the tensile strength of concrete splitting of the beam end occurs. An example of this is shown in Fig. 37. The actual form of splitting of the beam end depends mainly on the side cover of the hook, and also to a certain extent on the bottom and end cover.

Since the resistance to splitting depends on the tensile strength of concrete it is clear that the quality of the concrete is a factor in the strength of the beam: the destructive action of the hooks can be lessened by the use of high strength concrete. The same result can be achieved also in other ways. In particular, the provision of spiral reinforcement

in the end of the beam and the increase of the side cover of the hooks result in an apparent increase in the resistance of concrete.

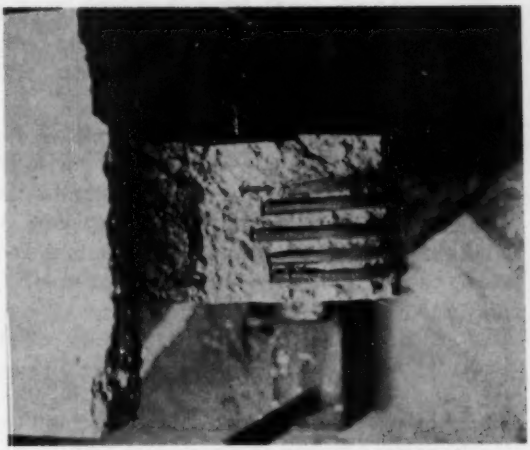


Fig. 36 — Top view of movement of hooks in a T-beam at collapse (flange has been removed)

An indication of the effect of spiral binding can be obtained from Graf and Brenner's[45] pull-out tests on 1-in. diameter plain bars. When the specimens were made of 1270-psi concrete the pull-out forces for plain bars, for bars with standard hooks, and for bars with hooks and a spiral (0.197-in. diameter wire at 1-in. pitch) were in the ratio of 1:1.45:2.15. For 3570-psi concrete the corresponding ratios were 1:1.15:190, respectively.

The destructive action of the hooks can also be reduced by an increase in the area over which the hooks act. This can be achieved in a number of ways. The simplest way is to increase the radius of the hook, and the influence of this factor can be judged from Graf and Brenner's[47] later pull-out tests on plain bars. Taking the ratio of the internal diameter of the hook to the diameter of the bar as k, the values of the pull-out force were in the ratio of 1:1.23:1.59 for k of 2.5, 5, and 8, respectively. The average concrete strength in these tests was 2500 psi. For 4800-psi concrete the ratio of the pull-out forces was 1:1.22 for k of 2.5 and 8.

Fig. 37—Splitting of a beam end

A second way to increase the area over which the hooks act is by the provision of transverse bars on the inside of the hook at its mid-

point and on the outside of the straight part of the bar beyond the hook. The transverse bars reduce the pressure of the concrete and, in addition, act as tension reinforcement and prevent the splitting of the beam end.

It could be expected from the pressure diagram of Fig. 35 that a further increase in the area over which the hooks act could be obtained by lengthening the straight portion of the bar beyond the hook.

The behavior of hooks on bent-up bars is similar to that of hooks on the tension steel; some relevant test results were given in Part 3 in connection with the influence of bent-up bars on the strength of reinforced concrete beams.

BOND STRESS

From the foregoing it can be seen that the problem of bond is very closely related to the shear-tension failure of reinforced concrete beams. In fact, in the past it has not always been clear whether it is the diagonal tension or the bond per se that is responsible for what appears to be a shear failure.

In reinforced concrete design the bond stress is checked, and the permissible value of this stress, according to the ACI,[33] British,[34] German,[35] and many other codes, is related to the compressive strength of concrete.

The actual physical determination of the strength in bond of a given steel and a given concrete is difficult. The oldest and generally used test is the pull-out test. The conditions of this test, however, do not correspond to those existing in a beam in flexure. Here, both the steel and the concrete are in tension while in the pull-out test the concrete is in compression and the steel in tension. A modification overcoming this disability has been suggested by Hajnal-Kónyi[48] whereby the concrete could be put in tension through being anchored by additional reinforcement. A further point has been raised by Leonhardt[49] who observed that stresses normal to the reinforcing bar, although usually neglected, seriously affect the distribution of bond stresses.

A more modern test of bond strength is performed on special test beams with parts of the tension steel unbonded. This test has recently been adopted by the ACI,[50] and requires no further description here.

In both these basic types of bond tests the ultimate bond stress at failure is measured, as well as the value of the bond stress when a slip of a given magnitude takes place. A correlation of the bond strength measured in a test with the bond strength of similar steel in an actual beam is difficult, and indeed such comparisons are not generally attempted.

CALCULATION OF THE BOND STRESS IN A BEAM

In a flexural member the bond stress is calculated from a formula of the type:

$$u = \frac{V}{jd\Sigma o} \qquad (10)$$

This clearly shows the relation between the bond stress and the shearing force at the section considered. It follows also that the bond stress is zero in the part of the beam subjected to pure bending. It is known, however, that when flexure cracking takes place in a zone of pure bending the bond stress there is not zero, and it is the magnitude of the bond stress that affects the width and spacing of the cracks. Nonetheless, the highest bond stresses, as well as bond failure, occur where the shearing force is largest.

FACTORS AFFECTING BOND STRESS AT FAILURE

Eq. (10) is based on two assumptions:

(a) Concrete takes no flexure tension.

(b) The change in the tension force in the main steel between two sections is proportional to the change in the bending moment between these sections.

These assumptions will now be briefly investigated.

The first assumption is clearly inapplicable as long as the concrete at the level of the steel is uncracked. Such concrete is subjected to a tensile stress and, as a consequence, the actual bond stress is less than calculated.

When a crack has opened the force in the steel crossing the crack represents an external force relative to the embedded portions of the steel. This crack may be either due to flexure or to shear, but only the latter is of interest in the present context. To illustrate this situation data on some relevant beam tests are given in Table 30. T-beams 1 to 6 all had the same tension reinforcement without hooks but differed in the web width and presence of stirrups. Although they all failed due to the slip of steel and a consequent loss of composite action, the failure of different beams took place under different ultimate loads. This means that, since the value of Σ_o was the same for all the beams and jd very nearly the same, *the ultimate bond stress in these beams depended on factors other than those of Eq. (10).*

Beams 1 to 3 differ in the width of the web only and it follows, therefore, that the width of the web, or some factor dependent on this width, influences the load carrying capacity of the beam and consequently the ultimate bond stress. The same argument can be applied to Beams 4 to 6 with vertical stirrups, again differing in the width of the web only.

The failure crack pattern of Beams 1 to 3 is of the type described in detail in Part 1. The redistribution of internal forces is followed by the opening of a horizontal crack at the level of the tension steel between the bottom of the diagonal tension crack and the support, and it is the formation of this crack that has destroyed the composite action of the beam and led to its failure. But the opening of the diagonal tension crack, which led to the formation of the horizontal split, depends on the width of the web as this affects the area over which the principal tension acts: the wider the web the higher the load under which the diagonal tension crack opens. Now, the later this crack opens the later starts the destructive action of the force in the main steel at its intersection with the crack. Thus the splitting of the concrete at the level of the tension steel is delayed with a consequent increase in the ultimate strength of the beam.

This argument is confirmed by a direct proportionality between the ultimate loads of Beams 1 to 3 and the widths of their webs.

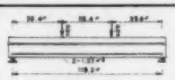
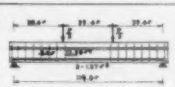
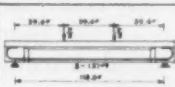
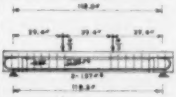
It should be emphasized that *it is the widening of the diagonal tension crack that is the primary cause of failure of the beam; the horizontal splitting of the concrete and the bond failure are only secondary phenomena* of shear-tension failure.

Table 30 shows that Beams 4 to 6 had a higher ultimate strength than Beams 1 to 3, and this can be attributed only to the presence of web reinforcement in the former group of beams. The role of this reinforcement can be best studied from the crack patterns of these beams: no horizontal split has been observed but only a number of fine cracks at the level of the steel since the stirrups prevented the pressing down of the tension steel, and the composite action of the beam was preserved. And even when the slip at the end of the main steel was observed, the load-carrying capacity of the beam was not yet exhausted as, owing to the presence of the stirrups, there was sufficient friction between the main steel and the surrounding concrete for a further increase in load on the beam to be taken.

The effect of the width of the web in Beams 4 to 6 was similar to its effect in the beams without web reinforcement except that the ultimate load of Beam 6 was rather lower than would be expected. The reason for this is that the bond failure of this beam started from a flexure crack which had developed before the diagonal tension crack.

From the above descriptions of failure it can be seen that in all beams, both with and without web reinforcement, the primary cause of failure was the formation and the widening of the diagonal tension cracks. As noted earlier, this is followed by the redistribution of internal forces and a slip of the tension steel. As a consequence, the tension force in steel between the bottom of the diagonal tension crack and the support no longer corresponds to the bending moment over that portion of the beam. It is, therefore, apparent that, once the

TABLE 30—BOND STRESS IN T-BEAMS WITH WEBS OF DIFFERENT WIDTH

Investigator	Beam No.	BEAM DETAILS	$\frac{a}{d}$	f_{cu} (a) psi	p percentage	r percentage	P_b kips	Σ_o psi	τ_b psi	$\frac{\tau_b}{f_{cu}}$	P_b kips	Σ_o psi	τ_b psi	$\frac{\tau_b}{f_{cu}}$	P_b kips	Σ_o psi	τ_b psi	$\frac{\tau_b}{f_{cu}}$	Mode of Failure
Bach and Graf [32]	1		2.9	3440	3.6	0	26.4	108	135	0.039	35.3	143	182	0.053	28.9	146	185	0.054	Pressing down and slip of tension steel
	2	As No.1 but web 5.9 in. wide	2.9	3440	4.8	0	22.0	90	154	0.045	27.1	110	188	0.055	27.1	110	188	0.055	
	3	As No.1 but web 11.8 in. wide	2.9	3440	2.4	0					50.0	203	169	0.049	54.0	219	187	0.053	
	4		2.9	3440	3.6	0.28	26.4	108	135	0.039	44.1	179	226	0.066	48.5	199	248	0.072	
	5	As No.4 but web 5.9 in. wide	2.9	3440	4.8	0.34	22.0	90	154	0.045	36.8	149	253	0.074	36.8	149	253	0.074	Slip of tension steel
	6	As No.4 but web 11.8 in. wide	2.9	3440	2.4	0.17					55.7	225	188	0.055	60.2	244	204	0.059	
	7		2.9	3440	3.6	0					48.5	196			54.5	221	286	0.083	Destruction of beam and by hooks
	12		2.9	3440	3.6	0.26					59.5	242			78.4	322	403	0.117	

Notes: (a) Measured on 7.9 in. cubes
(b) Based on width of stem
All steel consists of plain round bars

diagonal tension crack has formed, the assumptions on which Eq. (10) is based are not satisfied, and the equation does not give the correct picture of bond stress.

Beams 7 and 12 are included in Table 30 for comparison with Beams 1 and 4, respectively, the sole difference between the pairs of beams being the presence of hooks in Beams 7 and 12 and their absence in 1 and 4.

The bond stresses at failure of Beams 7 and 12 were 221 and 322 psi, respectively, while according to the current theory there should have been no difference in the bond stress of the two beams as the values of Σo and jd were the same in both.

To investigate the slip of tension reinforcement at failure five T-beams selected from those tested by Bach and Graf[18] will now be studied. The details of these beams are given in Table 31. All these T-beams had main reinforcement of plain round bars without hooks, and web reinforcement consisting of bent-up bars. The loading system was similar to that of Beams 1 to 6, and so was the percentage area of steel at midspan but the size and number of bars were suited to the bending-up arrangement.

Beam 28 was based on the single-lattice girder with the first member (reckoned from the support) in compression. Beams 35 and 37 were based on the double-lattice girder, and 39 and 41 on the triple-lattice.

The anchorage of the tension steel in this series of beams was better than that of the Beams 1 to 6 since part of the steel, being bent-up,

was anchored in the compression zone. For this reason the loads at which the first slip occurred, as well as the ultimate loads of these beams, were all higher than the corresponding values for Beams 1 to 6.

From Table 31 it can be seen that Beam 28 failed owing to the formation of a diagonal tension crack, followed by the pressing down of the main steel at the bottom end of the crack, splitting of the concrete at the level of the steel, and a slip of the steel, i.e., this beam exhibited a behavior similar to that of Beams 1 to 3 of Table 30.

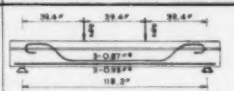
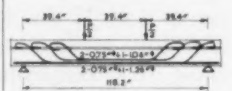
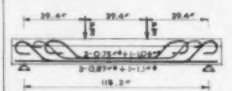
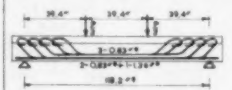
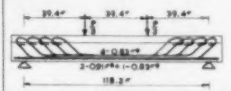
The initial diagonal tension cracks in the Beams 35 to 41 appeared under the same load as in Beam 28. With increase in load a number of additional diagonal cracks opened in the outer part of each beam. As the reinforcement against shear was good in this part of the beam, all the diagonal tension cracks remained fine. The pressing down of the tension steel and the splitting of concrete did not take place and, except for the straight bars, the composite action of the beam was preserved up to the end of the test. Owing to these circumstances the slip of the straight bars led to an increase in the tension force in the remaining bars, and the beam eventually failed by the yield of steel at midspan followed by the crushing of the concrete above. The ultimate load was lower than that of the beam with hooks as the absence of hooks resulted in a higher stress in the tension steel.

It is interesting to compare Beams 35 and 39 on the one hand and 37 and 41 on the other. In the former pair only one 1.26-in. diameter bar continued up to the support while in the latter there were two straight bars, 0.87- and 0.91-in. diameter, respectively. In these beams both the slip of steel and the ultimate failure occurred at considerably higher loads than was the case in beams with only one straight bar throughout. It is thus apparent that the distribution of the tension steel is of considerable influence on the load-carrying capacity of a beam.

It is instructive further to consider the efficacy of the web reinforcement in the beams of Table 31. The Σo of the straight bars in Beam 28 was twice that of Beams 35 and 39. It is not surprising, therefore, that slip at the end of the straight bars in the latter two beams was observed under a lower load than in Beam 28. However, Beam 28 collapsed due to "bond failure" while Beams 35 and 39 failed due to flexure-tension at midspan under a considerably higher load than the ultimate load of Beam 28. The explanation of this behavior is in terms of the efficiency of the web reinforcement; this was insufficient in Beam 28 while Beams 35 and 39 had satisfactory web reinforcement.

It may be mentioned that according to the German Code[35] the bond stress in bars smaller than 1.02 in. in diameter need not be considered in design, presumably because in such bars bond is never a critical factor. And yet, as shown above, "bond failure" occurred in Beam 28 with three 0.95-in. straight bars, while No. 35 and 39, with one 1.26 in. straight bar, failed in flexure.

TABLE 31—SLIP OF MAIN STEEL IN T-BEAMS WITH BENT-UP BARS

Investigator	Beam No.	BEAM DETAILS	$\frac{a}{d}$	f_{cu} (a) psi	p (a-a) percent	P_b kips	v_b psi	P_u kips	v_u psi	Mode of failure
Bach and Graf (18)	28		2.9	3440	3.6	66.2	319	77.5	374	Pressing down of steel and slip of straight bars
	35		2.9	3440	3.6	55.8	590	90.4	955	Slip of straight bars and yield of remaining steel at midspan
	37		2.9	3440	3.6	72.0	546	98.5	748	Slip of straight bars and yield of remaining steel at midspan
	39		2.9	3440	3.6	63.3	670	93.0	980	Slip of straight bars and yield of remaining steel at midspan
	41		2.9	3440	3.6	77.9	569	98.4	717	Slip of straight bars and yield of remaining steel at midspan

Notes: (a) Measured on 7.9 in. cubes
(a-a) Based on width of stem
All steel consists of plain round bars

Let us now consider the nominal bond stress, as calculated by Eq. (10), at the ultimate load of the beams listed in Tables 30 and 31. The values of this stress are between 110 psi for Beam 2 and 980 psi for Beam 39. The respective values of the bond stress corresponding to the load under which the first slip of the steel at the beam end was observed were 110 and 670 psi. It is clear, therefore, that the bond stress calculated by means of Eq. (10) is not representative of the actual bond stress in the beam, nor are the two stresses simply related. For this reason it is not possible to obtain an assurance of the composite action of the beam purely by limiting the value of nominal bond stress.

These observations, coupled with the proof given earlier that the assumptions of Eq. (10) do not hold good either before or after the beam has cracked, show that the use of this expression for design purposes is not satisfactory.

BOND FAILURE OR SHEAR FAILURE?

An interesting proof that the horizontal splitting of concrete at the level of the main steel is not caused by the bond failure but is a consequence of the widening of the diagonal tension crack, is offered by some tests of Bernander's.[19] He used 8 x 4-in. rectangular beams with a steel area ratio p of 1.2 percent but without web reinforcement. Deformed cold drawn high-tensile steel with various types of deforma-

tion (in rib area and spacing) was used. These different bond characteristics were found not to influence the ultimate shear strength of Bernander's beams, all of which failed in shear tension.

To investigate the influence of anchorage on the shear strength of reinforced concrete beams he tested also beams under third-point loading with the main steel having different bond lengths beyond the support. This was achieved by each beam being tested twice. In the first test, the beam was supported near the ends over a 6 ft 11 in. span and subjected to third-point loading, with the a/d ratio of 4, until failure in shear-tension or flat-slope splitting took place. The remaining intact two-thirds of the beam were then tested under a central point load over a span of 4 ft 7 in., so that the value of the a/d was unaltered. Under this arrangement the tension steel in the middle third of the original beam was now anchored securely over the length of the overhanging part of the beam.

A number of beams were tested, and shear-tension failure was observed to occur equally frequently in the middle third of the original beam and in the part which has been adjacent to the support in both tests. It is evident, therefore, that it was not bond that was the initial reason of failure.

It is interesting to note that many of the beams tested by Bernander,[19] particularly when reinforced with very high tensile steel, failed at loads corresponding to a low nominal shearing stress. In some cases this stress would yield a factor of safety of less than two in relation to the permissible shearing stress of 114 psi prescribed by the Swedish Code of Practice. If the requirements of the ACI Code were taken as the criterion of design then Beams 3, 6, and 8 would have a factor of safety in shear of less than two.

When no web reinforcement is provided, from the moment shear-tension failure has started to develop, the behavior of beams with deformed bars is little different from the behavior of beams with plain bars: once the diagonal tension crack has reached the level of the tension steel, this is pressed down and splitting occurs, leading to shear-tension failure. The conditions making a beam prone to shear-tension failure were described in Part 1.

On the other hand, when stirrups are present horizontal splitting is prevented (see Part 2) and the higher bond resistance of the deformed bars can prevent slip until the beam fails in some other way, possibly by yield of steel at midspan.

Let us now summarize the conclusions from the tests of Tables 30 and 31, both of which deal with beams failing in shear-tension, commonly considered as "bond failures." It can be seen that in beams with inadequate shear reinforcement the primary cause of failure was the formation and widening of the diagonal tension crack leading to the slip of the main steel. This constituted the immediate cause of failure

but it did not occur at a constant value of the bond stress for different beams. It does not seem possible, therefore, to talk about a fixed value of bond strength for a given concrete, steel, and loading conditions. The permissible value of the bond stress for a given factor of safety cannot be fixed either. There seems to be thus little point in checking a beam design for bond but it is essential to provide a satisfactory reinforcement against shear.

CONCLUSIONS

It has been shown that when the web reinforcement of a reinforced concrete beam exhibiting incipient shear-tension failure is inadequate, hooks at the ends of the main steel, consisting of plain bars, can materially increase the load-carrying capacity of the beam. This increase is greater the less sufficient the web reinforcement of the beam.

At the collapse of a beam in shear-tension the hooks disrupt the beam ends but this destructive action of the hooks can be lessened by an increase in the strength of concrete, by provision of spiral reinforcement in the end of the beam, by increase of cover to the hooks, or by increase in the area over which the hooks act.

Hooks at the ends of bent-up bars have a similar effect to that of hooks at the ends of the tension steel. Here too, the importance of good anchorage is greater the less sufficient the web reinforcement.

Although numerous beams with deformed bars have been tested for shear strength in recent years, nearly all of them were of rectangular cross section and practically all failed in shear-compression. From such tests no conclusions on the role of hooks at the ends of deformed bars can be drawn. It is believed that *when vertical stirrups are present,* some beams with plain round bars may fail in shear-tension while beams with deformed bars are much more likely to fail in shear-compression since this type of reinforcement offers greater resistance to slip and consequently composite action in the end parts of the beam is better preserved.

In T-beams with deformed bars the large compression zone makes the shear-tension failure more likely but sufficient data are not available to assess quantitatively the role of hooks when various types of deformed bars are used. It seems that with this type of reinforcement the hooks are very much less important than when plain bars are used. However, tests on T-beams reinforced by deformed bars with welded anchorage plates have shown that such anchorage has a considerable influence on the load-carrying capacity of the beam. Since T-beams are extensively used in construction further studies of this problem are believed to be of topical interest.

It has been shown that in normal beams bond failure is never a primary cause of failure of a reinforced concrete member subjected

to both flexure and shear but merely a consequence of the redistribution of internal forces following the widening of the diagonal tension crack.

No fixed value of the ultimate bond stress can be assigned to beams with different shear reinforcement, all other conditions being the same. For this reason, the use of a fixed permissible bond stress results in a greatly varying factor of safety. It follows that the calculation of bond stresses in a beam is not necessary, provided the beam is efficiently reinforced against shear failure.

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*References are numbered consecutively beginning with Part 1 of this five-part series.

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Discussion of the entire series (five parts) should reach ACI headquarters in triplicate by Mar. 1, 1961, for publication in June 1961 JOURNAL.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Construction

Strengthening works to a reinforced concrete coaling wharf

CHARLES G. CUMMING, *Proceedings*, Institution of Civil Engineers (London), V. 13 (Session 1958-59), May 1959, pp. 73-80*
Reviewed by ARON L. MIRSKY

Old wharf, built in 1907 of Hennebique "ferroconcrete," was strengthened to take load of new and heavier cranes by addition of new piles integrated with original structure by prepacked concrete.

*An abstract; original paper is on file in the Institution.

Slip forming is a winter job

RALPH MONSON, *Contractors and Engineers*, V. 57, No. 10, Oct. 1960, pp. 36-38, 42

Raising a slip form at a slow rate to compensate for slow set in cold weather, plus keeping concrete warm with heaters, made a winter job possible on cement storage silos.

The slip form was raised at a slower rate than usual; maximum movement was 16 ft in 24 hr. Specifications required keeping the concrete warm for 24 hr after placement. The contractor hung a 16 ft wide shroud of polyethylene film from the slip form and placed 20 gas burning heaters inside it. Nine additional salamanders were placed inside the bins at ground level. Temperature inside the silos seldom dropped below 70 F during the 14 days of continuous concreting. Under the shroud on the outside, the temperature did get down as low as 45 F when the outdoor temperature was 8 below zero.

The tanks were 36 ft in inside diameter and 150 ft high. The walls were 9 in. thick. A second cylinder with 12-in. walls was cast inside each of the main silos to a height of 65 ft to support the steel bin bottoms. This was cast as a separate slip-form operation in each tank after the main tank walls were completed.

Design

Inverse design of beams and grillages

JACQUES HEYMAN, *Proceedings*, Institution of Civil Engineers (London), V. 13, (Session 1958-59), July, 1959, pp. 339-352, Discussion, May, 1960, pp. 115-117
Reviewed by ARON L. MIRSKY

Instead of proceeding from an assumed moment diagram (expressed in terms of the unknown redundants) to the elastic curve of the deflected structure, which involves assuming the cross-sectional dimensions of the members, and then evaluating the redundants by use of the boundary conditions, the proposed method starts with an assumed deflected structure whose equation is expressed as $\delta = f(x)$ and which satisfies the boundary conditions; this is differentiated twice to give the curvatures and hence the bending moments; with the aid of the equilibrium equations, the flexural rigidities EI are then evaluated and the cross section dimensions established to suit the required rigidities and the maximum allowable stresses. Members with variable depth are thus easily handled. Two

A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 6, Dec. 1960, *Proceedings* V. 57. Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are *not* available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

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examples are given — a simply-supported reinforced concrete beam, and a propped cantilever steel beam.

Proposed method is then extended to the elastic design of grillages, based on the small deflection theory of flat plates with transverse loadings. Two examples are given, a floor of infinite extent supported by a regular array of columns, and a clamped circular grillage.

Application of the method to the plastic design of grillages is also discussed.

Protection from moisture for slab-on-ground construction and for habitable spaces below grade

NAS-NRC Publication No. 707, Building Research Advisory Board, 66 pp. $1.50 (Available from Printing and Publication Office, National Academy of Sciences National Research Council, Washington, D. C.)

Review of conditions giving rise to potentially unsatisfactory conditions in concrete slabs-on-ground and habitable spaces below grade. Recommendations are made for slab-on-grade design according to moisture conditions and also a guide to selecting allowable flooring materials. Similarly recommendations are included for moisture protection for exterior walls below grade based on soil conditions. A number of research studies are recommended to determine moisture transmission through slabs-on-ground, measuring moisture in concrete slabs, properties of coatings and membranes, and field investigations of actual performance of structures of this type.

Materials

Special reinforcement of reinforced concrete and reinforcement (wires and bars) for prestressing [Les armatures speciales de beton arme et les armatures (fils et barres) de precontrainte]

Bulletin, Centre D'Etudes de Recherches et D'Essais Scientifiques (Liege), Tome Special, 1958, 948 pp.

A collection of papers in French, German, or English on special reinforcement for reinforced concrete, wire and bar reinforcement for prestressed concrete, and methods of anchorage and connections. Each section is preceded by the summary of the general reporter for the particular theme.

Prof. Georg Wästlund, Stockholm, Sweden, was general reporter on special reinforcement and added to material presented on properties of American, Belgium, and German steels, pertinent data on Norwegian and Swedish steels. Various requirements for bond test were discussed along with the possibilities of brittle failure with hard grade steel, proper bending radius, bond between concrete and steel, cracking, deformations, and shear considerations.

Prof. H. Louis, University of Liége, Belgium, was general reporter on steels for prestressed concrete. Papers represented the viewpoint of the manufacturer of the steel and another group the views of the user. Prestressing steels discussed were those used in the as-rolled condition, heat-treated steel without cold working, and heat-treated, cold-worked steel. The reporter noted that the symposium did not aim at establishing specifications for prestressing steels but, nevertheless, sufficient data had been presented to develop more than a skeleton specification.

The third symposium theme, on anchorage, bond, and connections, was reported by Prof. Hubert Rüsch, Munich, Germany. The five papers presented considered both reinforced

and prestressed concrete. Anchorage by bond and by hooks in reinforced concrete were compared and the requirements for special anchoring devices and end blocks in prestressed concrete reviewed. Lap splicing and welding with emphasis on weldability, or lack of it, in high strength steel is also discussed.

Precast Concrete

Precast pile-columns put shopping center a story above grade

Engineering News Record, V. 164, No. 8, Feb. 25, 1960, pp. 40-42, 46, 48

Building an entire shopping center on piles, which extend some 20 ft above the ground surface and serve as one-story columns, makes Honolulu's giant Ala Moana Shopping Center unique. The reasons for placing the shopping center and an upper parking deck one story above the street level parking area were poor subsoil conditions and a high ground water level. The pile-columns were prestressed and precast in two basic sizes: (1) 18-in. diameter columns with a design load of 125 tons (these support buildings at mall level), and (2) 16 ½-in. diameter columns with a design load of 74 tons (these support the parking deck). Pile driving operations and construction procedures are described.

Prestressed Concrete

Facts on precast-prestressed folded plate slabs

HARRY H. EDWARDS, *Modern Concrete*, V. 24, No. 2, June 1960, pp. 42-46

Following a brief description of how folded plates work, article lists advantages and future possibilities of precast-prestressed folded plate slabs. Among the advantages listed are, weight reduction and material savings this type of construction affords. Machine production and special types of surface finishes are among the future possibilities noted. A short discussion of the design of folded plates is included.

Techniques for more economical prestressed beams

JAMES N. ANTILL, *Constructional Review* (Sydney), V. 33, No. 3, Mar. 1960, pp. 26-28

Greater economy is obtained in prestressed beams if the prestressing moment is varied along the length of the span in order to make full use of the maximum permissible fiber stressed at mid span as well as at the ends. The four techniques—combined pre- and post-tensioning, variable cross section, draping prestressing cables, and bond prevention—are treated briefly.

Socoman prestressed pipeline in South Africa

The Engineer (London), V. 209, No. 5427, Jan. 29, 1960, pp. 200-201
Reviewed by ARON L. MIRSKY

Brief illustrated description of the Socoman-Freyssinet prestressed pipe and its theory, and of the laying of a line of this pipe from the Umgeni Dam to Durban alongside an existing steel pipeline.

Average concrete stress along prestressing steel in prestressed concrete beams

KENNETH KAI-DIAO, *Journal*, Prestressed Concrete Institute, V. 5, No. 2, June 1960, pp. 61-68

In estimating the loss of prestress due to elastic shortening and creep effect of concrete in prestressed concrete beams, it is necessary to calculate the average direct stress in concrete at the level of the prestressing steel. For straight, prismatic, simple beams with undraped pretensioning strands, the average concrete stress along the center of gravity of steel due to a uniformly distributed load over the whole span is simply two-thirds of the value at the midspan. However, for haunched beams, continuous spans, cantilevers, and beams with curved prestressing units, the evaluation of such stresses may become quite cumbersome. In this article, rational yet simple formulas for computing the average direct stress in concrete along parabolic post-tensioning tendons in simply supported beams

are derived. The underlying principle of this procedure may be readily applied to other more complicated cases. Similar formulas for computing the average direct stress in concrete along the center of gravity of steel of straight strands are also presented for the ready use in engineering offices.

Tentative recommended practices for grouting post-tensioned prestressed concrete

PCI Committee on Grouting of Post-Tensioned Tendons, *Journal*, Prestressed Concrete Institute V. 5, No. 2, June 1960, pp. 78-81

The purpose of grouting is to provide permanent protection to the post-tensioning steel and to develop the designed ultimate strength of the structural member. The report covers definition of terms, materials, ducts, equipment, mixing, placing, cold weather work, and testing.

Properties of Concrete

Investigation of aggregates for concrete with reference to risk of reactions between alkalies and silica (in Swedish)

H. Hagerman and Heino Roosaar, *Nordisk Betong* (Stockholm), V. 4, No. 2, 1960, pp. 119-141

Reviewed by Margaret Corbin

Reports on studies carried out in Sweden since 1954 on harmful effects due to the chemical nature of aggregate. The studies consisted of petrographic examinations, chemical analyses (ASTM C 289-54T) and mortar bar expansion tests (ASTM C 227-52T). Earlier investigations in Denmark had revealed expansion effects due to flint concretions in chalk deposits. Similar formations exist in Scania in southern Sweden and have yielded similar results. A 5 percent admixture of flint to granitic material produced an expansion of slightly over 0.3 percent after 12 months. The alkali content of the cement was 1.2 percent of equivalent Na_2O. Bars made with aggregates obtained from rock types belonging to the Caledonian (Sweden)series did not exhibit cracks or any formation of alkali-silica gel.

Sonic modulus as related to strength and static modulus of high strength concrete

M. R. Sharma and B. L. Gupta, *Indian Concrete Journal* (Bombay), V. 34, No. 4, Apr. 1960, pp. 139-141

Research workers are evincing a progressively greater interest in the sonic method of testing concrete. Static modulus is largely used to compute the deflections of concrete members. The sonic or dynamic modulus is suitable only to give instantaneous deflections. These two moduli should therefore be correlated over a wide range of concrete strengths in order to afford a wider field of utility to either modulus. This paper reports work undertaken to obtain correlations between the sonic modulus of concrete and its strength characteristics as well as its static modulus over a range of compressive strengths between 3000 and 8000 psi. Strength characteristics considered are compressive strength and modulus of rupture. Article reviews previous work and recommends formulas for correlation of the moduli.

Structural Research

Sudden failure by snapping of clamped concrete slab with two-way reinforcement (in Swedish)

Eddy Abrahamsson, *Nordisk Betong* (Stockholm), V. 4, No. 2, 1960, pp. 101-108

Reviewed by Margaret Corbin

It has been observed in tests of plain and reinforced concrete slabs restrained at the edges that blast loading resulted in ultimate loads considerably higher than those attributable to a dynamic strength increase alone. This was ascribed to an "arching" effect, followed by a sudden "snapping" failure. The article describes a static test undertaken to verify this assumption, in which a restrained (clamped) concrete slab was fitted as one of the walls of a pressure chamber containing compressed air. The observed cracking load was about twice the computed ultimate load of a clamped slab—and was followed by a sudden snapping collapse.

Some factors in the shear strength of reinforced concrete beams

A. M. Neville and E. Lord, *The Structural Engineer* (London), V. 38, No. 7, July 1960 pp. 213-223

Authors' Summary

The different types of shear failure of reinforced concrete rectangular and T-beams are described to show that the shear strength depends on factors additional to those recognized by the design formulas. Results of experiments on the influence on shear strength of the size of the compression zone of the beam and of hooks on plain and deformed bars are presented. Tests on rectangular beams with varying amounts of compression reinforcement suggest that this does not affect the shear capacity of the beam.

Result of structural test on folded plate roof

P. S. Balint, *Constructional Review* (Sydney), V. 33, No. 3, Mar. 1960, pp. 21-25

Reports structural tests on Blakehurst high school roof. Loads were applied by releasing some of the pipe scaffolding and recording results of the imposed loads on strain gages. Tests established that the 96 x 60-ft roof acted as a unit. Unsymmetrical release of dead weight did cause anticipated torsional effects as from an unfavorable wind load.

Preliminary study of ultimate load moment-shear interaction in reinforced concrete beams

F. A. Hammond and R. B. L. Smith, *Civil Engineering and Public Works Review* (London), V. 55, No. 647, June 1960, pp. 792-794

Problems are discussed which arise from recent research into the effects of shear on the moment of resistance of reinforced concrete beams. Experiments are described involving: (a) the double testing of beams to give a flexural reference value for each specimen; (b) the testing of sets of identical model beams to investigate the effect of varying moment-shear ratio.

The shape of the moment-shear interaction curve as given in the paper indicates that the reduction of flexural strength caused by shear effects tends to be very marked for small increases in shear above the critical value and therefore, a principle of ultimate load design should be to avoid such types of failure by the provision of sufficient web reinforcement.

Some structural aspects of the design of concrete pipelines

Noel W. B. Clarke and Oliver C. Young, *Proceedings*, Institution of Civil Engineers (London), V. 14, Session 1958-59, Sept. 1959, pp. 67-96; discussion, May 1960, pp. 90-105

Reviewed by Aron L. Mirsky

Reports results and conclusions to date of investigation by Building Research Station, D.S.I.R. Investigation has involved study of some recent failures, laboratory tests, and a literature survey. Major conclusion is that current British codes and standards pertaining to the design and construction of concrete sewer lines are in urgent need of revision. Adoption of Marston's method of load estimation is suggested; his formulas, graphs, and suggested load factors for various bedding conditions are reproduced in an appendix (pp. 89-96).

General

Highway and airport engineering

Adrian R. Legault, Prentice-Hall, Inc., New York, 1960, 483 pp., $11.65

Book is intended to serve as a text for introductory courses in highway and airport engineering. Divided into two parts, the first part gives a background of knowledge concerning the development, planning, construction, and use of highways and airports principally from the viewpoint of a civil engineer. Part II sketches briefly some of the developments and problems in other fields of cargo moving.

Author attempts to present the material in such a way that the interest of the students will be stimulated beyond what is found in the book. A selected bibliography is provided at the end of each chapter as an aid to

more comprehensive coverage when this is desired. Data on recent practices and developments from various organizations has been incorporated.

Exposed-aggregate panels — A new vogue

Contractors and Engineers, V. 57, No. 6, June 1960, pp. 64-67

Describes briefly the various stages of manufacturing exposed-aggregate panels, such as casting beds, placement and selection of aggregates, placement of concrete layers, removal of panels from mold, and craftsmanship.

Place of digital computers in civil engineering

ROBERT K. LIVESLEY, *Proceedings*, Institution of Civil Engineers (London), V. 15, Session 1959-60, Jan. 1960, pp. 15-34

Reviewed by ARON L. MIRSKY

Discusses development and use of computers in civil engineering practice. Illustrative examples drawn from three fields (elastic analysis of square floor slabs with edge beams, analysis and design of steel frameworks, and design of a divided-lane superhighway) are concerned with the selection and use of the appropriate approach and programming, rather than with numerical results (for which the reader is referred to various published papers, by author and others).

Sign supports: Foundation design

Bulletin No. 247, Highway Research Board, Apr. 1960, 35 pp., $0.80

Contains three papers presented at the 38th annual meeting of the Highway Research Board:

"Foundations to Resist Tilting Moments Imposed on Upright Cantilevers Supporting Highway Signs," by W. C. Anderson, analyzes the problem by selecting a foundation of given dimensions in a given soil, choosing a neutral axis, and then by integrating the soil resistance, both shear and moments are imposed.

"Tests of Tilting Moment Resistance of Cylindrical Reinforced Concrete Foundations for Overhead Sign Supports," by F. E. Behn, describes the construction and testing of cylindrical foundations of reinforced concrete approximately 32 in. in diameter, 8 and 12 ft deep in three soil types classified as plastic, granular, and organic.

"A Device for Evaluating Horizontal Soil Resistance for Overhead Sign Supports," by W. C. Anderson, describes the apparatus consisting of two partial sections of a cylinder each 12 in. long and 4 in. wide, with a 6-ton hydraulic jack between. The device is lowered in an 8-in. auger hole at desired depths. initial readings are taken, then additional pressure is applied until the desired deflection is reached.

Testing concrete and mortar in tension

D. P. O'CLERY and J. G. BYRNE, *Engineering* (London), V. 189, No. 4900, Mar. 18, 1960, pp. 384-385

Reviewed by ARON L. MIRSKY

Describes a simple, easy-to-use grip based on the lazy-tongs principle and gives some results of tests performed with it. Results are very consistent; none had to be rejected because of specimen failure due to nonaxial loading. Authors suggest that tension test is more representative of concrete quality than compression test and that tension tests, using these grips, be used for on-site testing.

Conversion for compressive strengths of concrete cubes and cylinders (in Swedish)

JUKKA VUORINEN, *Nordisk Betong* (Stockholm), V. 4, No. 2, 1960, pp. 143-154

Reviewed by MARGARET CORBIN

Earlier investigations are reviewed and a formula suggested for practical use: $K/S = 1.60 - S/45$, wherein K is the compressive strength of 20-cm cubes in kg per sq cm; S—the compressive strength of 15 x 30-cm cylinders in kg per sq cm. The conversion formula is verified on a job site. Another series compares 20-cm cubes with 20 x 20-cm cylinders and comes to the conclusion that equivalent compressive strengths are the result.

ACI NEWS LETTER

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memo from the President:

COMPETITION OR COOPERATION?

"Of making books there is no end." Likewise, there seems to be no end to forming organizations devoted to a particular cause. We are living in an organization society. And properly so, since organizations have accomplished far more than would have been possible through uncoordinated efforts of individuals.

There are organizations of engineers, architects, and contractors. There are associations of producers of such materials and services as cement, aggregate, reinforcement, admixtures, ready-mixed concrete, masonry, pipe, prestressing, testing, and so on. There are societies concerned with materials specifications and testing, construction specifications, highways, railways, bridges, dams, education, research, standards, building codes, and other important aspects of the construction industry.

Some sincere and thoughtful persons have expressed concern that the construction industry is trending toward over-organization; that there are so many splinter groups that attention is divided, participation is limited, and results are duplicated if not actually in conflict. They fear that ACI programs will be hindered because of jurisdictional problems, or that ACI coverage is so broad that it will in some respects be meaningless.

Your President does not share these views. He believes that through proper cooperation and coordination the many will accomplish more than the few. Some duplication of ACI work is unavoidable; in fact, it has long existed.

But differing and even opposing viewpoints are bound eventually to result the adoption of the best; and even competition would be preferable to complacency.

How does ACI relate to other organizations? There is no corresponding organization for other construction materials, wherein many users and individuals join with producers and dealers as a unified membership. Nor is ACI a professional society or a trade association. In a sense, ACI is an umbrella organization, bringing together the people and interests of many specialized fields onto the common ground of concrete. ACI does not conflict with ASCE, which is interested in concrete but only as one of many facets of that broad profession; in fact, there are six committees organized *jointly* by ACI and ASCE. Nor does ACI conflict with ASTM, which covers many materials in addition to those for concrete; ACI has official representation on several ASTM committees, and there is a clear and cordial understanding as to the respective fields of the two organizations. ACI also has an official representative on one or more committees of several other organizations.

This is cooperation, not competition. Every possible approach should be made to the problem of improving concrete structures and minimizing their cost. A certain amount of overlapping and duplication is bound to occur; but by coordination through appropriate committees and other means such as direct negotiation, it is possible to avoid jurisdictional conflicts and needless duplication. Actually, through cross-communication the overlapping activities can be made to strengthen the work of both organizations concerned.

Joel W. Kelly
President

IN THE PART 2 DECEMBER 1960 issue of the JOURNAL, which accompanies this issue, you will find—

Discussion of papers published in the April through June, 1960, JOURNALS

Errata and addenda conveniently compiled for the entire **Proceedings** V. 56

Contents listing papers published from July 1959 to June 1960, JOURNALS

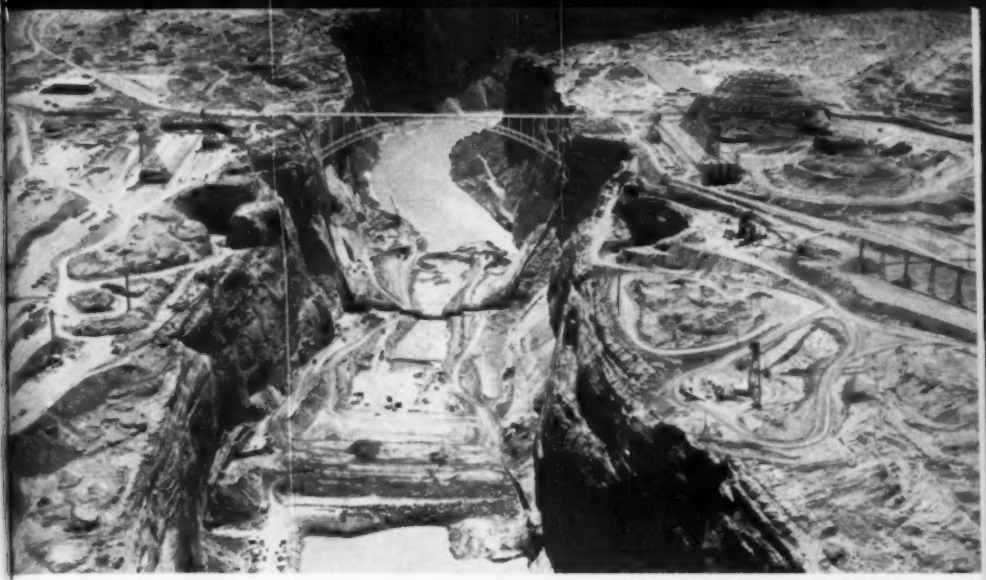
Indexes for the **Proceedings** and Current Reviews along with an index to the technical material published in the "News Letter" sections for the convenience of readers who retain the monthly issues.

Discussion of papers published in the July through September, 1959, JOURNALS appeared in March, 1960; discussion of papers published October through December, 1959, appeared in June 1960; discussion of papers published January through March, 1960, appeared in the Part 2, September, 1960, issue. Discussion pages in the Part 2 December 1960 issue are numbered continuously from the final page of the Part 2 September issue.

How to assemble into one volume—

Part 2 December 1960 issue now completes **Proceedings** V. 56. When JOURNALS for July 1959 through June 1960 and Part 2 September 1959 are disassembled (after removing wire stitches), covers, masthead (page just inside front cover), and News Letter sections (unless you prefer to retain these), the remaining sections may be assembled with the parts of Part 2 December to complete **Proceedings** V. 56. Start with title page and contents taken from Part 2 December and follow with the issues separated as noted above from July to June in chronological order. Then add discussion pages from Part 2 September and discussion pages and indexes from Part 2 December.

Complete Proceedings V. 56 is then in one convenient volume.



Photo—U.S. Bureau of Reclamation

World's Largest, Fastest Cableways Speed Concreting at Glen Canyon Dam

CONCRETING IS well underway at Glen Canyon Dam in Arizona at the site of what will be the nation's second highest dam. Concrete and concrete materials are described in detail on p. 629 of this month's JOURNAL.

Traveling on 50-ton cableways, two 12-cu yd buckets — the largest ever built for this application — are placing concrete at an average of 7500 cu yd per day.

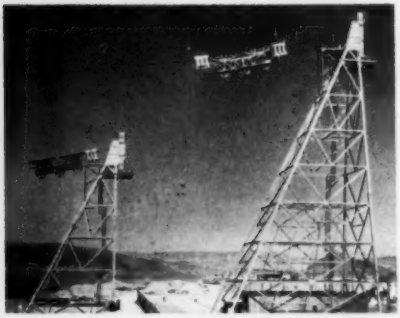
A tailormade electrical system, engineered by General Electric Co., powers the giant traveling cableways. The huge buckets are able to operate at speeds and efficiencies never before available for cableways. Maximum hoist speed of the cableways is 700 ft per min and maximum haul speed is 1400 ft per min, making it the fastest cableway ever constructed.

Operating at these rates, Merritt-Chapman & Scott Corp., prime contractor for the project, expects to complete the placing of 5,200,000 cu yd of concrete in 27 months.

A 25-ton cableway with an 8-cu yd bucket which has a hoist speed of 500-600 ft per min supplements the two 12-cu yd buckets at Glen Canyon. This provides an over-all capacity of 32 cu yd and a daily concreting rate of 9600 cu yd. Until now, the 25-ton cableway represented the largest system of its type used in dam construction.

By comparison, concreting at Hoover Dam required eight buckets — five handling 8 cu yd of concrete, and three handling 4 cu yd — with a total capacity of 52 cu yd. Average concreting rate was 6150 cu yd per day. Although operating at 38 percent less over-all bucket capacity, engineers estimate they will be able to place about 35 percent more concrete per day than the average at Hoover Dam, the nation's highest.

Both 50-ton cableways at Glen Canyon are equipped with movable head



Traveling carriage and outrigging platform at top of head tower on 50-ton cableways at Glen Canyon Dam

and tail towers located on opposite banks of the canyon and connected by a 4-in. main track cable. The concreting bucket is suspended from a traveling carriage which moves along the track cable. The cableways are constructed at different elevations so that one highline can pass directly over the other. The higher cable stretches 2050 ft while the lower unit extends 1800 ft.

Each tower is powered by four 40-hp a-c wound-rotor motor drives. A major feature of the cableway operation is the automatic skew correction control provided for tower movement. If one tower moves ahead of the other during concreting operations, a selsyn differential limit switch arrangement automatically retards the leading tower until realignment occurs.

Hoisting and hauling motions are completely independent of each other.

By using independent motor drives, each with five speed control points, overlapping of the hoist-haul motions is possible in an almost unlimited number of directions. This reduces time lost by the single motor, conventional reeving system that generally prevents overlapping of the hoist and haul motions and requires that they be performed in sequence rather than simultaneously.

Only one other d-c dual-drive cableway has been used in dam construction. That was an 8-cu yd, 25-ton rig used on Pine Flats Dam on Kings River near Fresno, Calif., in 1950.

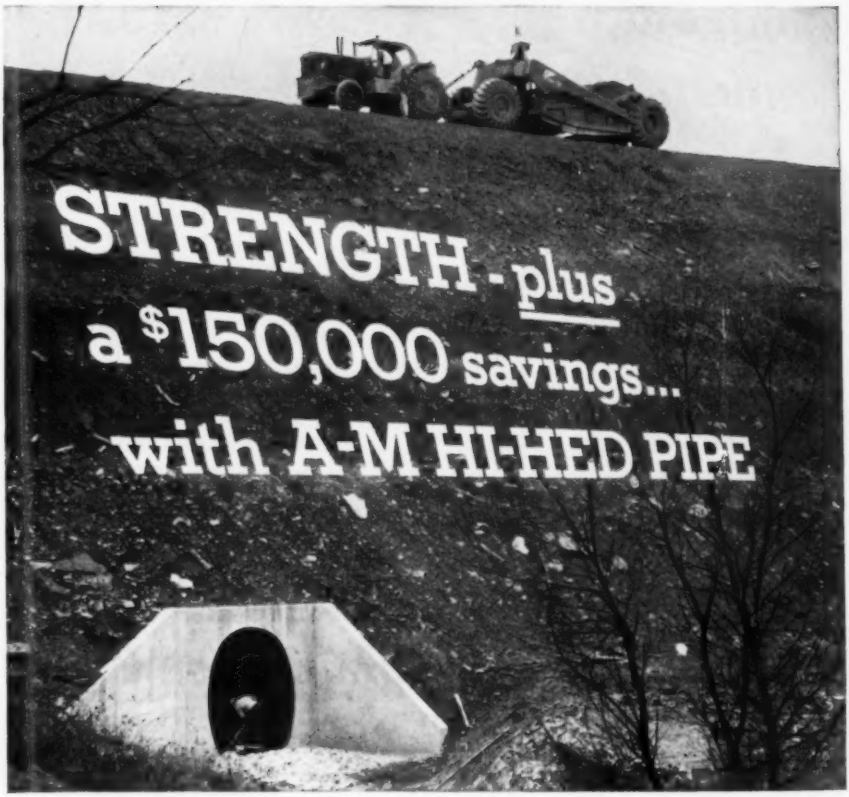
General Electric engineers said the d-c drives result in improved speed and torque control with less disturbance of power. Less braking and simpler operation are other advantages, they said.

Power for the d-c motors is derived from three generators mounted on a common motor generator with a 1250-hp a-c drive motor. The hoist drum is driven by two 500-hp motors powered by two 400-kw generators with the four armatures connected in series. A single motor and generator power the haul drum.

Design of Concrete Pavements

Recommended Practice for Design of Concrete Pavements (ACI 325-58) covers the design of rigid airport and highway pavements and bases for conditions of climate, traffic, available construction materials and equipment, and construction methods of the United States. Includes recommendations for soil foundations, selection of slab dimensions, joints, and details for reinforced or nonreinforced concrete. 36 pp. $1.00 per copy, 50¢ to ACI members.

Order from Publications Department, American Concrete Institute, P. O. Box 4754, Redford Station, Detroit 19, Michigan.



STRENGTH was the first requirement for a 4,000 foot drainage structure under the new east-west runway at Greater Pittsburgh Airport. Precast concrete Hi-Hed pipe provided this essential strength (50% stronger on the average than equivalent size round pipe), plus a big bonus in the form of an estimated saving of $150,000—approximately 18% savings over competitive material.

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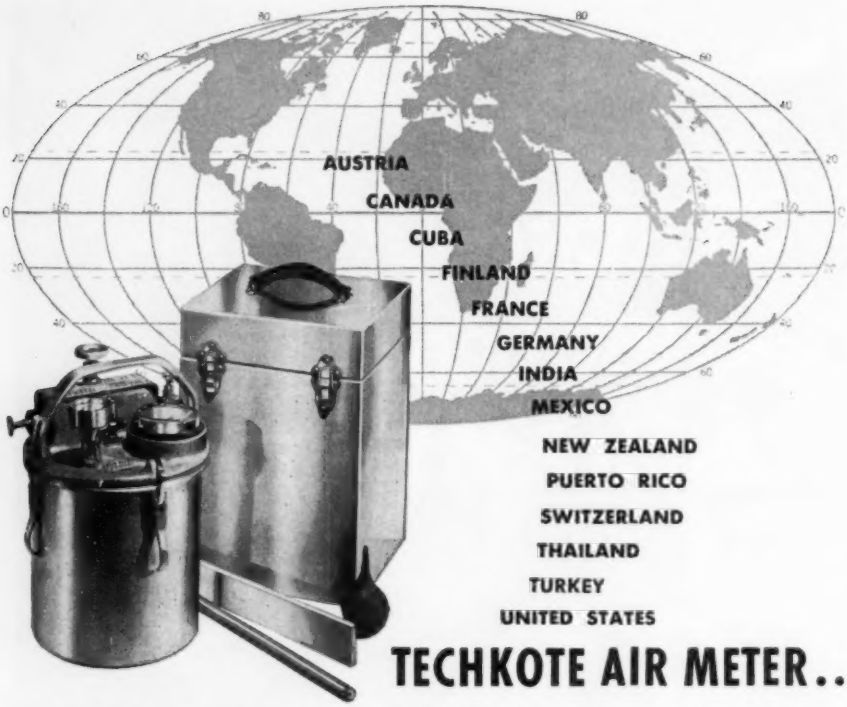

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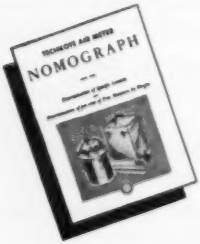

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TAC approves technical program for 1961 annual convention

The technical program for the 57th annual ACI meeting in St. Louis, February 20-23, at the Chase-Park Hotel, was set by the Technical Activities Committee at its meeting October 31 in Tucson during the 13th ACI regional meeting.

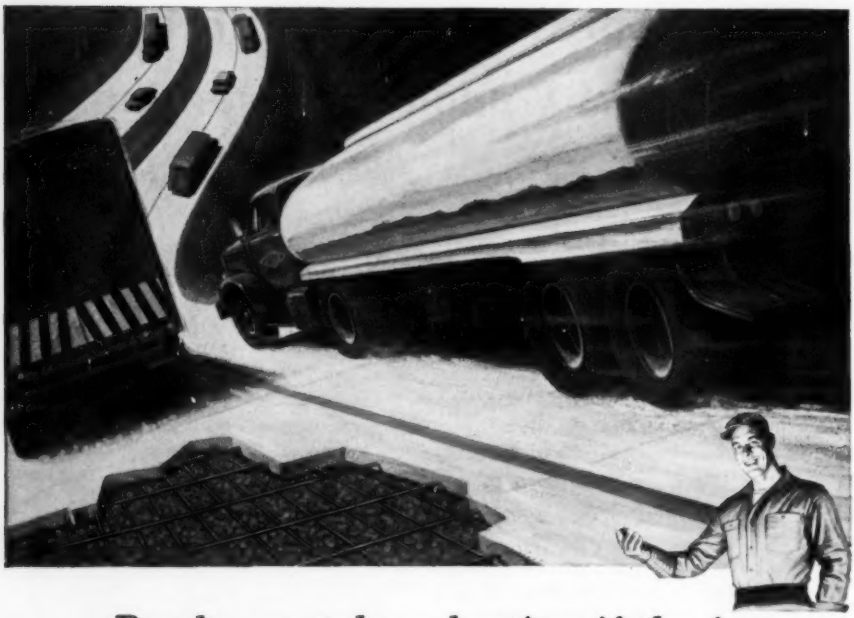
The first 1½ days of the convention will be devoted to technical committee meetings with a Technical Committee Progress Session on Tuesday afternoon, February 21. A number of brief committee reports will be presented at this session on work underway and progress within various committees.

Five technical sessions have been planned with several sessions running concurrently through Thursday, February 23. The first general session opens Wednesday morning following addresses by Mayor Raymond Tucker of St. Louis and ACI President Joe W. Kelly, Berkeley, Calif. The session will consist of reports summarizing recent European meetings of importance, including the meeting of the International Association of Bridge and Structural Engineers, and the Third International Congress of the Precast Concrete Industry. The session program also includes a report of the Autoclaved Cellular Concrete Conference and a report of recent tours of cement and concrete plants in Russia.

The technical session gets into full swing Wednesday afternoon with concurrent sessions on design and construction, and materials. The Design and Construction Session will feature four papers covering such subjects as: Mitchell Park Conservatory; prestressed and preloaded concrete pillars; analysis of stresses and the load carrying capacity of the Lincoln Boulevard Bridge over Lost River, Idaho; and supporting structure for retractable roof of Pittsburgh public auditorium. Four papers scheduled for the Materials Session will be based on water-cement ratio versus strength; chemical prestressing of concrete elements using expanding cements, load tests on patterned concrete masonry walls, and a discussion on "How Good is Good Enough?"

Concurrent meetings Thursday morning will feature the ACI Committee 216 Symposium on Fire Resistance as well as a design session. Papers to be presented at this design session will deal with theory and analysis of prismatic folded plate structures; controlled-deflection design method for reinforced concrete beams and slabs; rectangular concrete stress distribution in ultimate strength design; and tie requirements for reinforced concrete. Committee 115 will hold its annual research forum on Thursday afternoon. About 20 technical papers will be presented by prominent American and European concrete authorities during the four-day meeting.

Howard A. Coleman, vice-president and general sales manager, Missouri Portland Cement Co., St. Louis, is general chairman of the local committee which is arranging an exhibit to be held in conjunction with the convention, a ladies program, and a number of "extras" to make the 57th annual ACI meeting thoroughly worthwhile.



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Positions and Projects

ACI technical committee appointments

Listed below are committee members who have recently accepted appointments to ACI technical committees. Included are *new* appointments only.

Committee 116, Admixtures
Aron L. Mirsky
University of Connecticut
Storrs, Conn.

Committee 208, Bond Stress
Sabri Sami
West Virginia University
Morgantown, W. Va.

Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete
Carl M. Rollins
Basalt Rock Co.
Napa, Calif.

Committee 215, Fatigue of Concrete
John W. Murdock
University of Illinois
Urbana, Ill.

Committee 216, Fireproofing or Fire Protection of Structures
C. M. Knudsen
Riverside Cement Co.
Riverside, Calif.

Committee 318, Standard Building Code
William Eipel
Construction Engineer
New York, N. Y.

Committee 333, Design and Construction of Composite Structures—Joint ACI-ASCE
Joseph H. Appleton
University of Alabama
Birmingham, Ala.

Committee 338, Torsion
Henry J. Cowan
The University of Sydney
Sydney, Australia

Committee 339, Allowable Stresses in Reinforcement
Roger Diaz de Cossio
University of Mexico
Mexico, D.F., Mexico

Committee 611, Inspection of Concrete
John J. Manning
The Concrete Industry Board, Inc.
New York, N. Y.

Committee 612, Recommended Practice for Curing Concrete
Otto C. Frei
Georgia Lightweight Aggregate Co.
Atlanta, Ga.
J. J. Shideler
Portland Cement Association
Skokie, Ill.

Committee 712, Precast Structural Concrete Design and Construction—Joint ACI-ASCE
James R. Gaston
Fountain Sand and Gravel Co.
Pueblo, Colo.
Samuel L. Selvaggio
American Marietta Co.
Chicago, Ill.

Committee 716, High Pressure Steam Curing
D. L. Sutter
Anchor Concrete Products Co., Inc.
Buffalo, N. Y.

Committee 805, Application of Mortar by Pneumatic Pressure
J. J. Shideler
Portland Cement Association
Skokie, Ill.

ACI-ASCE Committee 333 releases report

ACI-ASCE Committee 333, Design and Construction of Composite Structures, presents a progress report on "Tentative Recommendations for the Design of Composite Beams and Girders for Buildings" on p. 609 of this month's JOURNAL.

The committee was organized in 1956 to prepare recommendations for the design and construction of structures composed of prefabricated beams combined with cast-in-place slabs. After a review of the existing information and practices, the committee channeled one part of its activities toward the preparation of this report.

The progress report is written in two parts. Tentative design recommendations are presented in the first part. The second part contains explanations of the provisions of the design recommendations.

I. M. Viest, bridge research engineer, AASHO Road Test, Ottawa, Ill., is committee chairman; R. S. Fountain, Portland Cement Association, Chicago, secretary.

Balance of the 18-man committee is comprised of I. A. Benjamin, director of research, Granco Steel Products Co., Granite City, Ill.; W. E. Bradbury, manager, Product Engineering and Development, Sheffield Steel Division, Armco Steel Corp., Kansas City; A. A. Brielmaier, professor, Department of Civil Engineering, Washington University, St. Louis, Mo.; G. C. Driscoll, Jr., professor, Department of Civil Engineering, Fritz Engineering Laboratory, Lehigh University, Bethlehem; M. E. Fiore, Parsons, Brinckerhoff, Quade and Douglas, Inc., New York; F. J. Hanrahan, executive vice-president, American Institute of Timber Construction, Washington, D. C.; N. W. Hanson, structural development engineer, Portland Cement Association, Chicago; and W. J. Jurkovich, senior bridge engineer, California Division of Highways, Sacramento.

Others working on the committee are N. J. Law, Mississippi Valley Structural Steel Co., St. Louis, Mo.; A. M. Lount, A. M. Lount and Associates, Toronto; James Michalos, professor, chairman, Department of Civil Engineering, New York University, New York; P. P. Page, Jr., Seeley, Stevenson, Value and Knecht, New York; B. Thurlimann, professor, Department of Civil Engineering, Swiss Federal Institute of Technology, Zurich; R. J. Van Epps, Portland Cement Association, Chicago; C. H. Westcott, consulting engineer, Westcott Engineering Co., Chicago; and Ardis White, professor, Department of Civil Engineering, University of Houston, Houston.

The direction and coordination of the efforts leading to the progress report were assigned to Philip P. Page, Jr.

The committee expects to prepare further reports after the completion of current studies.

Winter Concreting

RECOMMENDED PRACTICE FOR WINTER CONCRETING (604-56) advises the use of air-entrained concrete and addition of one percent of calcium chloride by weight of cement for cold weather concreting. These permit a reduction in the time newly placed concrete should be protected. Other factors covered in this ACI Standard include the use of accelerators and antifreezes, keeping of temperature records, heating of materials, subgrade preparation, protective coverings, heated enclosures, curing and form removal. 23 pp. Price 75¢, 50¢ to ACI members.

AMERICAN CONCRETE INSTITUTE PUBLICATIONS

ACI-ASCE Committee 323 control group meets

The first meeting of the control group of the newly reorganized Joint ACI-ASCE Committee 323, Prestressed Concrete, was held at Chicago, on Sept. 6, 1960 to initiate activity under the following mission:

"To continuously evaluate the technical status of prestressed concrete and to recommend to the joint sponsors the formation of any needed technical committees or subcommittees for the study of particular developments, innovations, or problems arising in this field. Such committee activity should be directed toward the publishing of information and the development of tentative recommendations or recommended practices which incorporate new procedures or correct existing deficiencies. The committee will also study significant research problems and make recommendations."

The control group assigned priorities to some 28 items that had been suggested for activity by the committee. The first priority items to be worked on by the committee are:

(1) Steam curing of prestressed concrete members

(2) Shear

(3) Use of entrained air in mixes for prestressed concrete

(4) Bond of pretensioning steel

(5) Deflection, camber, and vibration

(6) Re-evaluation of allowable stresses under working loads

(7) minimum percentages of steel

Several other second priority items will also receive attention in the near future. These are:

(1) End block stresses

(2) Evaluation of forces due to creep and shrinkage introduced into prestressed elements and their effects on the remaining elements of a structure

(3) Long prestressed concrete columns

(4) Torsion and stability

(5) Design concepts for prestressed plates and slabs

It was recognized by the control group that many of these items fall within the jurisdiction of certain existing committees of both ACI and ASCE. Therefore, these committees will be contacted by the subcommittee chairman prior to any activity by Committee 323. It is hoped that anyone having pertinent information on any of these items, will forward such information to the committee chairman or secretary.

Morris Schupack, Schupack and Zollman, Stamford, Conn. is chairman of the newly organized committee and W. Burr Bennett, Jr., Portland Cement Association, Chicago, is secretary. Members of the committee are: J. N. Clary, Virginia Department of Highways, Richmond; Ben C. Gerwick, Jr., Ben C. Gerwick, Inc., San Francisco, Calif.; Jack R. Janney, The Engineers Collaborative, Des Plaines, Ill.; T. Y. Lin, University of California, Berkeley; Alan H. Mattock, Portland Cement Association, Skokie, Ill.; George S. Pinter, Precrete, Inc., Corona, N.Y.; C. P. Siess, University of Illinois, Urbana; and W. R. Wilson, The Atchison, Topeka and Santa Fe Railway System, Chicago, Ill.

ACI headquarters viewed by many

During the past month over 100 guests have inspected the Institute headquarters building in Detroit. The largest contingent was a group of 54 architectural students from the University of Illinois, Urbana.

The building has been included in the itinerary of countless engineers, architects, and small groups of students passing through Detroit.

Northern California Chapter cosponsors fall meeting

A joint meeting cosponsored by ACI Northern California Chapter and Region 11, Construction Specifications Institute, and participated in by American Institute of Architects, Central Valley Chapter Structural Engineers of Central California and AGC of Northern and Central California was held at Sacramento Inn, Sacramento, on September 30.

The speaker, Walter Kunze, manager of the Structural Bureau, Portland Cement Association, Chicago, presented an illustrated lecture on "New Ideas in Architectural Concrete." The subject dealt with exposed concrete surfaces such as conventional types of plywood and such liners as rubber matting, plastics, Fiberglas, and others. Also colors in concrete surfaces in-

cluding integral coloring and the use of exposed aggregates through the use of aggregate transfer, precasting, and other techniques were illustrated, along with artificial aggregates such as glass, burned ceramics, and tile. The subject was of extreme interest and brought about many questions and discussion from the floor.

ACI President Joe W. Kelly and Vice-President Lewis H. Tuthill were present. Co-chairmen were George F. Winslow, chairman CSI, Region 11, and Harry E. Thomas, president of ACI Northern California Chapter.

ACI Oklahoma Chapter cosponsors conference

Oklahoma State University, Stillwater, presented a 3-day concrete course, on mix proportioning, November 2-4, in cooperation with the ACI Oklahoma Chapter and the Portland Cement Association. R. P. Witt, professor, School of Civil Engineering, Oklahoma State University, was conference director. Professor Witt is vice-president of the Oklahoma chapter and a member of ACI Committee 115, Research.

ACI President Joe W. Kelly, professor of civil engineering at the University of California, Berkeley, was featured speaker at a joint dinner meeting on November 3.

Course lecturers included Les W. Baxter, program coordinator for the course, general field engineer for the northeast Oklahoma district of PCA; George B. Southworth, director of training for the Master Builders Co., Cleveland; and James H. Sprouse, sales and service engineer for Master Builders Co., Oklahoma City branch. All of the lecturers are ACI members.

The intensive course covered subjects on the effect of ingredients on quality of concrete; concrete requirements for the job; analysis of concrete materials, proportioning of concrete mixes; and control and adjustment of concrete mixes.

Subsequent lectures covered recommended test methods; evaluation of admixtures; and related subjects such as transporting, placing, finishing, and curing of concrete. Pumped concrete, prestressed, post-stressed, slip-form, tilt-up, and lift slab were discussed by the speakers.

The Oklahoma chapter arranged the date of their fall meeting to coincide with the November 3 meeting of the conference.

Other chapter news from Stan Gerlach, McMichael Concrete Co., Tulsa, secretary-treasurer of the Oklahoma chapter, announces that James H. Sprouse, sales and service engineer, Master Builders Co., Oklahoma City branch, has been appointed to the chapter board of directors to fill the vacancy resulting from Harold Wenzel's departure from Oklahoma City.

Michigan chapter members tour Peerless plant

On the evening of October 14 a group of about 30 ACI Michigan Chapter members and guests were conducted on a tour through the manufacturing facilities of the Peerless Cement Co., Detroit. The tour was conducted by Peerless employees who are also members of the local chapter.

The tour was preceded by a showing of the Portland Cement Association film, "From Mountains to Microns." It was followed by a coffee break and discussion period.

Hawaiian Cement elected to PCA membership

The Hawaiian Cement Corp., Honolulu, has been elected to membership in the Portland Cement Association, Chicago.

PCA has established an Hawaii District to serve cement users in the 50th state. Joe V. Williams, Jr., formerly structural engineer in PCA's Texas District, has been named district engineer in Hawaii, with offices in Honolulu.

Neville participates in Northwestern U seminar

A. M. Neville, lecturer in engineering at the University of Manchester, Manchester, England, currently head of the Department of Civil Engineering, Nigerian College of Technology, Zaria, Nigeria, was featured speaker at an engineering seminar, October 13, conducted by Northwestern University. Dr. Neville spoke on creep properties of concrete and their effects.

The meeting was one of a series of graduate seminar and colloquim meetings sponsored by the engineering, chemistry, and physics departments of the university during October.

Toledo U conducts roof design course

A structural engineering short course in the design of reinforced concrete folded plate roofs was conducted at the University of Toledo, Toledo, Ohio, November 14.

The course was cosponsored by the University and the Portland Cement Association. D. E. Hoeffel, PCA structural engineer, Cincinnati, was instructor for the course, assisted by C. M. Brown, PCA structural engineer for the Toledo area. Messrs. Hoeffel and Brown are both ACI members.

NCMA magazine wins national award

Concrete Masonry *Pictorial*, published by the National Concrete Masonry Association, Washington, D.C., has received a major advertising award.

The colorful magazine, which promotes new architectural uses of concrete block locally under the auspices of many of the 700 members of NCMA, was judged a winner in the external house publication group of the annual Direct Mail Awards of the Direct Mail Advertising Association. *Pictorial* was selected for the award from more than 300 entries.

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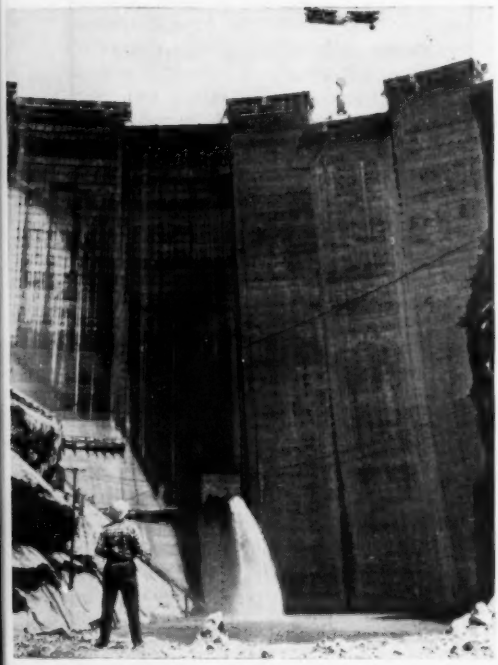
CONCRETE PERFORMANCE REPORT:

POZZOLITH concrete used in the Tri-Dam Project to meet full range of engineering requirements for all types of concrete specified . . . saves $86,000 in costs of concrete alone

The Tri-Dam Project in south-central California is one of the most interesting hydraulic programs undertaken in recent years. Each of the three structures represents a different basic type of dam being constructed in North America today.

Donnells Dam is a graceful concrete arch. Tulloch Dam is a concrete gravity structure and Beardsley Dam a rolled earth fill dam with a concrete spillway.

TRI-DAM



DONNELLS DAM rises 480 feet above foundation. This project included construction of a 7-mile long concrete-lined tunnel, penstock and powerhouse.

Shortly after the project began, laboratory and field tests were made to determine the quality and economy of POZZOLITH concrete—using cement and aggregates already in use on the project.

Results showed that the POZZOLITH Retarder mix would lower the rate of temperature rise in the mass concrete . . . reduce total heat evolved . . . provide other basic improvements in quality . . . and produce substantial savings in cost of concrete. Net savings were computed after allowing for the cost of additional aggregate required to maintain the same yield.

A total of over 515,000 cubic yards of concrete were used for tunnels, powerhouses and the dams themselves. POZZOLITH was used in 499,715 cubic yards of concrete to effectively aid in placing operations in all situations encountered. Strength requirements were met with substantial reduction in cost of concrete.

The net savings averaged $0.172 per cubic yard—a total of $86,000.

Concrete work at Tri-Dam extended through all seasons with temperatures ranging from 110°F to below freezing. Maximum aggregate sizes of the mixes ranged from ¾" to 6" and design strengths ranged from 2,000 to 3,000 psi at 28 days.

TULLOCH DAM stretches nearly ½ of a mile across the Stanislaus River. Here POZZOLITH Retarder proved particularly beneficial in mass concrete during summer months when the temperature soared to 110°F.

TRI-DAM FACILITY is jointly owned and operated by the Oakdale and the South San Joaquin Irrigation Districts, California. Design Engineers: George T. Goodall Co. & International Engineering Co., Inc. • Project Engineers: Tudor-Goodenough Engineers • Contractors: Donnells Dam and Beardsley Dam—Tri-Dam Contractors (a joint venture by Morrison-Knudsen Co., Peter Kiewit Son's Co., Macco Corp., and Stolte, Inc.); Tulloch Dam—The Arundel Corp. and L. E. Dixon Co.

PROJECT

Throughout the project, Master Builders field men assisted in solving on-the-job concrete problems . . . adding their experience and know-how to that of Tri-Dam Project engineers, contractors and concrete suppliers.

The local Master Builders field man will welcome discussing *your* requirements. Call him in to demonstrate how concrete produced with POZZOLITH is superior in quality, performance and economy to plain concrete or concrete made with any other admixture.

Write for your free copy of the detailed "Tri-Dam Concrete Performance Report".

BEARDSLEY DAM—an earth fill dam with concrete spillway. The structure is 1,000 ft. long and 280 ft. high.

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**POZZOLITH* is a registered trademark of The Master Builders Co. for its ingredient for concrete which provides maximum water reduction, controls rate of hardening and increases durability.*

Twelve ACI members honored by ASCE

Twelve ACI members were among 31 members of the American Society of Civil Engineers recently honored at the ASCE 1960 annual convention in Boston, Mass., October 10-14.

Glenn W. Holcomb, chairman of the civil engineering department at Oregon State College, Corvallis, became the 92nd president of ASCE. He is a former director and vice-president of the society and widely known as an engineering educator.

Henry W. Buck, senior partner in the firm of Buck and Buck, Hartford, Conn., was elected director for ASCE District 1. Earle T. Andrews, vice-president, Pennsylvania Glass Sand Corp., Hancock, W. Va., was elected director for District 6. Mr. Andrews is currently a member of ACI Committee 716, High Pressure Steam Curing. John B. Scalzi, Construction Section of the Market Development Division, United States Steel Corp., Cleveland, was elected director for District 9. Harmer E. Davis, professor of civil engineering as well as head of the Institute of Transportation and Traffic Engineering, University of California, Berkeley, was elected director for District 11. Long active in ACI activities, Professor Davis was a member of the Institute Board of Direction from 1949-51. He has served on various technical committees and has contributed generously to the pages of the JOURNAL.

Honorary members

Two ACI past presidents were elected to honorary membership by ASCE. Solomon C. Hollister, dean of the College of Engineering, Cornell University, Ithaca, and Frank Kerekes, dean of the faculty at Michigan College of Mining and Technology, Houghton.

Dean Hollister, an ACI member since 1917, has an international reputation as an engineering educator. He is also widely known for his researches in concrete and reinforced concrete. A prolific writer, he has contributed numerous papers for publication in the JOURNAL. Dean Hollister has participated in nearly every phase of Institute activity including serving as president of ACI in 1932.

Dean Kerekes, a distinguished engineering educator, has been an ACI member since 1938. He has always been an enthusiastic researcher, his principal interest lying in the field of elastic behavior of reinforced concrete and steel structures as determined by actual field tests and model analysis. He is author or coauthor of three textbooks and some 20 technical papers, many of which have been published in the JOURNAL. Dean Kerekes has served the Institute in many capacities on executive and technical committees. He was president of the Institute in 1956 and is currently a member of Committee 318, Standard Building Code.

Awards

Three ACI members were presented awards for ASCE *Transactions* papers published in 1959 which were considered important contributions to the advancement of the profession. The Collingwood Prize, which is restricted to younger members of the society, was awarded to Ralph L. Barnett, research engineer, Stanray Corp., Chicago, for his paper on "Prestressed Truss Beams." The Leon S. Moisseiff Award was presented Frank Baron, professor of civil engineering, University of California, Berkeley, and Harold S. Davis, structural engineer, General Electric Co., Richland, Wash., for their paper "Pressure Lines and Inelastic Buckling of Columns." Professor Baron is currently a member of ACI Committee 314, Rigid Frames for Buildings and Bridges, and Committee 334, Concrete Shell Structures.

Othmar H. Ammann, consulting engineer and partner in the firm of Ammann and Whitney, New York, was the recipient of the Ernest E. Howard Award. Mr. Ammann was honored "for design and construction of outstanding

bridges of record dimensions, and large engineering works, including the George Washington Bridge, the Lincoln Tunnel under the Hudson River, and the Narrows Bridge now under construction at the entrance to New York Harbor.

Joseph Peraino, chief engineer of the Construction Department, Merritt-Chapman and Scott Corp., New York, was awarded the Construction Engineering Prize, which is given for an outstanding *Civil Engineering* article. He has designed and worked on the installation of equipment and plant facilities for many outstanding projects, including the aggregate, batching, and cableway installations for Glen Canyon Dam, described in his prize-winning article in the June 1959 issue of *Civil Engineering*.

Corsell assumes Toch presidency

Peter P. Corsell, formerly associated with Sonneborn Chemical and Refining Corp., New York, has joined the firm of Toch Brothers, Inc., New York, as president and general manager.

PCA presents lecture series on hyperbolic paraboloids

The Portland Cement Association presented four weekly lectures on concrete hyperbolic paraboloids in cooperation with Wayne State University, Detroit, October 26, November 2, 9, and 16.

While the lectures dealt mainly with the theory, design, and construction of hyperbolic paraboloid shells, speakers also covered the design and finishing of industrial floors, the design of parking lots, and a discussion of latest information on concrete fire-resistance tests.

Dudley Newton, professor and head, Department of Civil Engineering, Wayne State University, acted as chairman of the series. Among the PCA men presenting the lectures were: J. Gardner Martin, Michigan district engineer, Lansing; W. C. Krell, supervising structural engineer, Michigan district; E. M. Pell, structural engineer, Grand Rapids; John Banker, regional construction superintendent, Chicago; and J. E. Kratzer, paving engineer, Detroit.

ACI *Proceedings* on microfilm

Institute members and JOURNAL subscribers may obtain complete ACI JOURNAL, V. 30, July 1958 through June 1959 (*Proceedings* V. 55), including discussion and indexes, on microfilm for $6.15.

This is an economical means of maintaining a file of ACI material with the minimum storage space.

Proceedings V. 46 (September 1949 to June 1950) through *Proceedings* V. 54 (July 1957 through June 1958) are also available.

For additional price information and orders, write University Microfilms, 313 N. First St., Ann Arbor, Mich.

Day promoted to new Idaho highway post

H. L. Day, formerly assistant materials engineer for the Idaho Department of Highways, Boise, has been made materials engineer for the department. He succeeds L. F. Erickson who was appointed assistant construction engineer.

McDevitt joins sales staff of Ford, Bacon and Davis

Lawrence J. McDevitt, a marketing and sales specialist for major steel companies for the past 15 years, has joined the staff of Ford, Bacon and Davis, Inc., New York consulting engineering firm.

Mr. McDevitt will be a supervising engineer specializing in sales and marketing analysis. He was with FB&D before World War II on a special assignment involving a study of a natural gas utility.

Reinforced Concrete Association announces meeting schedule

The Reinforced Concrete Association, London, has announced its meeting schedule for the 1960-61 session. The program lists meetings for London and the North-Western, the Midland Counties, the Scottish, the West of England, and the South Wales and Monmouthshire Branches.

The schedule covering October, 1960 through March, 1961, will include among its topics at the various meetings: nuclear reactor containment, parking garages, lift slab construction, research, prestressed concrete, ready-mixed concrete, and model analysis. Advance proofs of papers are made available 7 days prior to a meeting.

The Reinforced Concrete Association is dedicated to the advancement of reinforced concrete in which professional, scientific, and industrial interests are brought together to promote, encourage, and improve the art, science, and practice of reinforced concrete design and construction. Information on meetings of the association should be obtained from the Secretary, London, or the honorary secretary of any branch.

Errata

The following corrections should be made in "Internal Forces in Uniformly Loaded Helicoidal Girders", by A. C. Scordelis, which appeared in the April 1960, ACI JOURNAL.

p. 1016—there is a typographical omission in the equation given for δ_{zw}. Each term on the right-hand side of the equation should be multiplied by "sec α." In the actual computations for X_r and X_s it was included, so these values are correct as tabulated.

The following correction should be made in "Reactivity of Ultrafine Powders Produced from Siliceous Rocks," by K. M. Alexander, which appeared in the November, 1960, ACI JOURNAL.

p. 569—in Reference 10, change 1959 to 1955.

Huron to install huge rotary kiln at Alpena

Huron Portland Cement Co., Detroit, a subsidiary of National Gypsum Co., has announced plans to purchase a huge rotary kiln, 460 ft in length and 15 ft in diameter, for installation at its Alpena plant.

The new kiln and accessory equipment are expected to be in operation by January, 1962, increasing the plant's annual capacity by 2 million bbl. Daily output of the kiln will be 5250 bbl, 15 percent more than the combined capacity of the first eight kilns installed in the 53-year-old plant.

The Huron installation at Alpena has a present annual capacity of 12 million bbl.

Hersey elected ASTM district councilor

A. T. Hersey, director of technical service, Alpha Portland Cement Co., Easton, Pa., has been elected district councilor of the American Society for Testing Materials.

Long active in ACI committee work, Mr. Hersey is currently chairman of ACI Committee 331, Structures of Concrete Masonry Units, and a member of Committees 402, Concrete Floor Finishes, and 613, Recommended Practice for Proportioning Concrete Mixes.

Universal Atlas to erect St. Louis cement terminal

Universal Atlas Cement Division of U. S. Steel is prepared to start construction of a cement distributing station in St. Louis, Mo.

Present plans call for the erection of a marine terminal, cement storage silos and loading facilities for bulk and packaged cement at this location. A large self-unloading cement barge to serve the station will be built by American Bridge Division of U. S. Steel. The cement terminal will be supplied by barge and rail from Universal Atlas Cement's producing plant at Hannibal, Mo.

Figure to SAVE on placing, bracing and stripping

Dual Lane Bridge, Highway 85 near Greer, S. C. Contractor: La-Motte Construction Co. Designer and Engineer: South Carolina Highway Bridge Dept.

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LOOKING AHEAD

Dec. 12-15, 1960—First Industrial Building Exposition and Congress, New York Coliseum, New York

Jan. 5-7, 1961—FIP-RILEM Symposium on Injection Grout for Prestressed Concrete, Norges Tekniske Hogskole, Trondheim, Norway

Jan. 9-11, 1961—Annual Convention, Concrete Products Manufacturers Association of Quebec, Chateau Frontenac, Quebec City, Que., Canada

Jan. 9-13, 1961—Annual Meeting, Highway Research Board, Washington, D.C.

Jan. 17-19, 1961—44th Annual Convention, National Crushed Stone Association, Hotel Americana, Bal Harbour, Fla.

Jan. 17-19, 1961—Winter Conference and Exhibit, Instrument Society of America, Sheraton-Jefferson Hotel and Kiel Auditorium, St. Louis, Mo.

Jan. 18-20, 1961—Annual Convention, National Concrete Contractors Association, Diplomat Hotel, Hollywood-by-the-Sea, Fla.

Jan. 23-26, 1961 — Annual Plant Maintenance and Engineering Show, International Amphitheatre and Palmer House Hotel, Chicago, Ill.

Jan. 23-26, 1961—31st Annual Convention, National Ready Mixed Concrete Association, Hotel Americana, Bal Harbour, Fla.

Jan. 26-28, 1961—13th California Street and Highway Conference, Institute of Transportation and Traffic Engineering, and University Extension, University of California, Berkeley, Calif.

Jan. 30-Feb. 2, 1961—12th Biennial Concrete Industries Exposition, National Concrete Masonry Association, Cobo Hall, Detroit, Mich.

Feb. 20-23, 1961—57th Annual Convention, American Concrete Institute, Chase-Park Plaza Hotel, St. Louis, Missouri.

Western Waterproofing opens Tampa sales office

Western Waterproofing Co., St. Louis, has announced the opening of a sales office in Tampa which will handle all of the firm's weatherproofing and restoration projects in Florida. C. H. Wells is in charge of the office.

NCMA-ABPA merge

The National Concrete Masonry Association and Autoclave Building Products Association have effected a unification according to a joint announcement.

As a result of this action ABPA will become a new division of NCMA, responsible for research and development of autoclave curing. The division will be established as part of NCMA's Washington, D.C., headquarters.

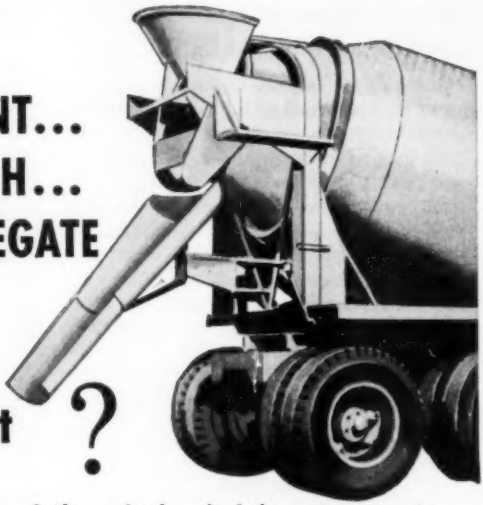
One of the first projects planned for this new division is a special 2-day session on autoclave curing which will take place during the 1961 NCMA convention in Detroit, January 30-February 2.

Marshall elected ASTM executive secretary

Thomas A. Marshall, Jr. has been elected executive secretary of the American Society for Testing Materials by its board of directors.

Fred F. Van Atta, formerly assistant secretary of ASTM, was elected to the post of treasurer with responsibility for the business operations of the society. Mr. Van Atta, a member of ACI since 1946, is a former staff member serving as assistant secretary and acting secretary-treasurer of the Institute. Robert J. Painter, formerly executive secretary and treasurer, will continue as consultant to the executive secretary. Raymond E. Hess will continue as associate executive secretary. As technical secretary and editor-in-chief, Mr. Hess is responsible for the technical activities and publications of ASTM.

How much CEMENT... How much FLY ASH... How much AGGREGATE ...go into the best mix for a given requirement ?

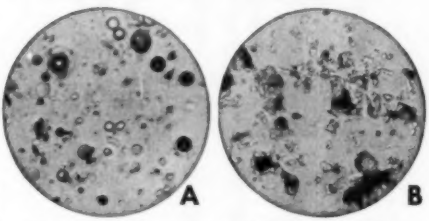


THAT correct design of the mix is vital is not open to question. The best results are obtained when ingredients are proportioned in accordance with information obtained from extensive investigation. Programs of this kind are continuing. The information developed is available through the companies named below.

Fly Ash Improves Quality in Several Ways

A great many tests have shown that a concrete containing proper amounts of an approved fly ash is superior to a concrete made without fly ash . . . Fly ash concrete is easier to place and finish, and has a lower heat of hydration. It also has greater resistance to the penetration of water and to the effects of aggressive waters.

All these are important, depending on the purpose of the concrete. One quality *always* desired is high workability, ease of placing in the forms, filling them completely, using even less water, with a consequently denser mix and more watertight job. Main reason for the high workability is seen in microscopic views A and B. The particles of cement (B) are rough, angular, while those of fly ash (A) are mostly spherical and smooth. Thus the "ball bearing" effect which makes the mix handle more easily.



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ISA schedules winter conference and exhibit

More than a dozen technical sessions have been scheduled by the Instrument Society of America for its Winter Instrument - Automation Conference and Exhibit to be held in St. Louis, January 17-19. All conference sessions will be held in the Sheraton-Jefferson Hotel, conference headquarters, while the exhibit, running concurrently with the conference, will be held in Kiel Auditorium. The St. Louis section of ISA, as host committee, is planning to make the meeting a memorable one.

At least three of the sessions will be "firsts" as part of an ISA conference program. These will be on biomedical instrumentation, cement and lime instrumentation, and a session under the auspices of the newly formed Measurement Standards Division of ISA.

The Cement and Lime Instrumentation session will feature technical papers on instrumentation and control techniques as applied to the rotary kiln, meters and controls for automated operation of rotary kilns, and the acceptance and advantages of instrumentation and automatic controls on rotary kilns. The papers will be presented from a user's point of view, and the session is being directed at management and operation for stabilizing quality of product, increasing production, and the cutting of fuel cost.

HELP SCIENCE FIGHT TB

ANSWER YOUR CHRISTMAS SEAL LETTER TODAY

Klein announces retirement

William H. Klein, executive vice-president in charge of production, Dragon Cement Co., New York, has announced his retirement as of October 1.

Argonne announces two engineering appointments

Chauncey H. Ladd and John O'Leary have been appointed engineers in the Plant Engineering Division of Argonne National Laboratory, Argonne, Ill.

In his new position Mr. Ladd will prepare plans and specifications for new buildings and for modifications of existing buildings. He was a structural engineer for Henschien, Everds, and Crombie, architects and engineers, Chicago, prior to joining Argonne.

John O'Leary will work on design and construction of the zero gradient synchrotron, a 12.5 billion electron volt particle accelerator now under construction at the Argonne site. Mr. O'Leary was associated with Kaiser Engineers, Chicago, prior to his Argonne appointment.

Harold F. Clemmer

Long-time ACI member Harold F. Clemmer, 72, who served for 29 years as engineer of materials for the District of Columbia Highway Department, died October 5.

Harold F. Clemmer

Mr. Clemmer came to the District in 1930, and retired from Government service last year. Since then he had been a consultant to the Highway Department and to other groups.

Born in Hampton, Iowa, he received BS and CE degrees from Iowa State College and later served on the staff, progressing to associate professor of civil engineering.

Mr. Clemmer entered the highway engineering field early in his career as an engineer of materials for the Illinois Highway Department. Later, he served as technical adviser to the Solvay Process Co. in New York before going to Washington, D.C.

An ACI member since 1937, Mr. Clemmer served as chairman of Committee 617, Specifications and Recommended Practice for Concrete Pavements and Bases, for 21 years. A number of his technical papers have been published in the ACI JOURNAL.

Mr. Clemmer was active in a number of other technical societies including ASTM, AASHO, ASCE, ARBA and Highway Research Board. He served as chairman of numerous committees within these organizations, and was the author of many technical articles, presented at their meetings.

Albert Haertlein

Albert Haertlein, associate dean of engineering and applied physics at Harvard University, recently died at the age of 64.

A 30-year Institute member, Professor Haertlein was a distinguished educator and authority on the mechanics of structures. A graduate of Harvard University, he joined the Harvard faculty in 1928. He had been associate dean since 1951. He had been a member of the Board of Registration of Professional Engineers and Land Surveyors of the Commonwealth of Massachusetts since 1942, serving several times as chairman.

Professor Haertlein had served ASCE as director and vice-president, and became honorary member in 1956. Last February he was given an award of merit by the Engineering Societies of New England "in recognition of his long and outstanding service to the engineering profession."

Professor Haertlein held an honorary degree from Northeastern University, the Desmond Fitzgerald Medal of the Boston Society of Civil Engineers, and the award of merit of the National Council of State Boards of Engineering Examiners.

Harry N. Howe

Harry N. Howe, Memphis structural engineer almost 53 years and a 50-year ACI member, died recently at the age of 78. He designed and supervised much of the building that helped Memphis grow to the metropolis of the Mid-South and was recognized as one of the outstanding men in his field in the South. He had also contributed much to the civic and political life of Memphis.

Born in Fulton, N. Y., Mr. Howe grew up in Washington, D.C. and graduated as a civil engineer from Cornell University in 1904.

He first came to Memphis with the U. S. Engineers assigned to survey parties but returned to New York City in 1906 to become associated with Turner Construction Co., one of the pioneers in the reinforced concrete building-sphere. The following year he returned to Memphis to start the independent engineering firm of Gardner and Howe. After his partner's death, Mr. Howe took over and later was joined by his son, Warner Howe,

who has continued in the business with him.

Among some of the later projects designed by the firm were such structures as Mississippi State Office Building, University of Mississippi Medical School and Teaching Hospital at Jackson, and the Madison East Unit of Baptist Hospital.

In 1951 he was appointed engineering member of Tennessee State Board of Architectural and Engineering Examiners. Mr. Howe was active in a number of engineering societies. He had been a corporate member of ASCE since 1910 and a life member since 1945. He was a charter member of Memphis Engineers Club and twice served as its president. In 1957 he was named Engineer of the Year by Memphis Chapter of the Tennessee Society of Professional Engineers of which he was a charter member.

Permanente's Hawaii plant now in production

The new $13 million facility at Waianae, Oahu, replaces Permanente's 20-year old system of transporting bulk cement to the Island in company vessels from California.

Ultra-modern process employed at the plant involves a 450-ft long kiln; nuclear metering of material flow, closed-circuit television inspection of operations, and elaborate electronic controls and dust collection equipment. Rated annual productive capacity of the facility is 1,700,000 bbl of cement. It is among the few cement plants in the world using coral limerock as its basic raw material.

Plant buildings were designed with an eye to beauty as well as industrial function. The plant is a study in the versatility of modern construction with concrete.

Computer program for barrel shells

The firm of Ketchum, Konkel and Hastings, Denver, has developed an electronic computer program for calculation of stresses in barrel shell roof structures.

The program is for an IBM 650 machine with double the usual memory (4000 words instead of 2000). Almost any type of edge beam or any ratio of width to span can be solved and the structure may be symmetrical or unsymmetrical.

Running time for the symmetrical case is about 6 min and for the unsymmetrical about 30 min. The program was developed around the theory given in Gibson and Cooper's *Design of Cylindrical Shell Roofs* published by D. Van Nostrand. The differential equation is the well known DKS equation.

FATIGUE OF CONCRETE

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This bibliography lists and annotates 114 significant works published since 1898 on fatigue tests of plain and reinforced concrete. Titles cover a wide range of tests, including results of compressive and flexural fatigue loading and of resistance of bond to fatigue loading. 38 pp., 8½x11-in. format, punched for 3-ring binder.

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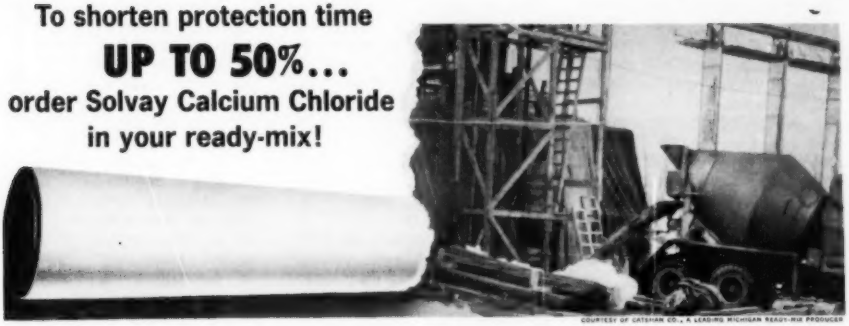

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Honor Roll

HONOR ROLL

Jan. 1-Oct. 31, 1960

Nearly 350 "member conscious" ACI'ers are listed on this month's Honor Roll. Why don't you sign up one new member this month? Tell your colleagues about ACI activities! Let's keep the membership spiraling.

Point System

1 point for Student; 2 points for Junior; 3 points for Individual; 4 points for Corporation; and 5 points for Contributing.

Samuel Hobbs	38
Alfonso Marin E.	33
Joaquin Spinel L.	20
Robert P. Witt	20
Archie E. Hjerpe	17
Walter H. Price	16
Roger H. Corbetta	15
Harry Ellsberg	15
Russell Porter	14
C. P. Siess	14
Faraj Tajirian	13
Antonio A. Henson B.	12
W. E. Moulton	12
James A. McCarthy	11
J. Raymond Watson	10
Jacob J. Creskoff	9
Napoleon Ferrer G.	9
Alfonso Golderos	9
Gene M. Nordby	9
Leslie E. Robertson	9
George B. Southworth	9
Milton H. Zara	9
James Chinn	8
Martin J. Gutzwiller	8
H. C. Pfannkuche	8
J. F. Toppler	8
Yoshikatsu Tsuboi	8
Felix C. Villoslada	7
W. S. Cottingham	7
Evan S. David	7
Ward W. Engle	7
Narbey Khachaturian	7
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J. Karni	6½
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The Board of Direction approved applications in the following categories: 65 Individual, 4 Corporation, 1 Contributing, 8 Junior, and 17 Student, making a total of 95 new members. Considering losses due to deaths, resignations, and nonpayment of dues, the total membership now stands at 10,307.

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APTULSKY, J. W., Haddon Heights, N. J. (Proj. Engr., Publicker Ind.)

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BELLIER, JEAN, Paris, France (Cons. Engr.)

BOUCHLAS, THALES, Chicago, Ill. (Soils Engr., Soil Test. Sers., Inc.)

CAHILL, JOHN E., Maracaibo, Venezuela (Engr., Creole Petr. Corp.)

DIFFENBAUGH, J. D., Riverside, Calif. (Gen. Contr.)

DOUGHERTY, EDWARD M., Nashville, Tenn. (Pres., Constr. Engrg., Inc.)

DOUGRAMEJI, M., Baghdad, Iraq (Asst. Prof., Univ. of Baghdad)

FINNIMORE, J. R., Wenatchee, Wash. (Sr. Fld. Engr., Stone & Webster Engrg. Corp.)

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FREEBURN, WILLIAM D., S. Holland, Ill. (Struct. Engr., Eugene A. Dubin, Cons. Engr.)

GALE, ADOLFO, Colon, Panama (Engr., Colon Free Zone)

GALLO DEZA, EDUARDO, Lima, Peru (Struct. Engr.)

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GREIG, JOHN M., Lewiston, N. Y. (Des. Engr., Merritt-Chapman & Scott)

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HJERPE, A. E., Chicago, Ill. (Vice-Pres., Gen. P. C. Co.)

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LEVI, FRANCO, Torino, Italy (Prof., Inst. Scienza delle Costruzioni-Politecnico)
LIU, ERNEST T. L., Seattle, Wash. (Struct. Engr., Worthington, Skilling, Helle & Jackson)
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MANICONE, NICHOLAS, Jersey City, N.J. (Struct. Engr., Grad, Urbahn, Seelye)
MATHUR, GANGESHWAR DAYAL, Lucknow, India (Supt. Engr., Govt. of U. P., India)
MINGESZ, JOHN, West Allis, Wis. (Struct. Engr., Mingesz Engrg. Co.)
MORALES, ABEL, Bogota, Colombia (C. E., Bavaria, S. A.)
MURRAY, ROBERT LESLIE, Gisborne, New Zealand (Const. Supv., Gisborne City Council)
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Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Post-tensioning services for prestressed concrete

Joseph T. Ryerson & Son, Inc., Chicago, one of the country's oldest steel firms, has entered the prestressed concrete field, supplying post-tensioning assemblies and field service equipment at job sites anywhere in the country.

Ryerson has pointed out that the post-tensioning method of developing prestressed concrete lends itself particularly well to on-the-spot construction of large members. Contractors can economically precast or cast-in-place on the job site, sections that are too big to be practically or economically cast elsewhere and shipped to the site.

In explaining its services which Ryerson said strongly support the use of post-tensioned prestressed concrete, it was indicated that they include, in addition to fabrication of steel tendons and anchor plates, force development calculations, detailing and placement plans, field labor procedures, and the furnishing of stressing and grouting equipment.

Ryerson uses the BBRV system of post-tensioning which was developed in Europe and which has been used there successfully for many years. The system permits all wires within a single metal conduit to be stressed simultaneously, and provides safe, positive anchorage.

Complete information about the company's post-tensioning facilities for prestressed concrete are available.—Joseph T. Ryerson & Son, Inc., Box 8000-A, Chicago 80, Ill.

Epoxy floor topping

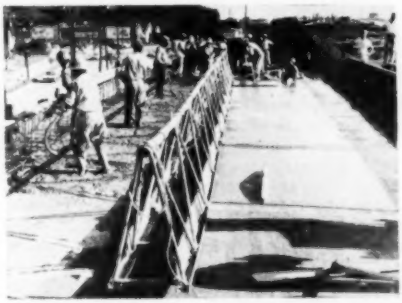
Thiopoxy 60 is a two-component floor topping and patching compound developed to withstand severe corrosion and abrasive conditions which tend to destroy or disintegrate ordinary concrete.

It is a one-application compound applied by trowel, to a ⅛ in. thickness or more, and finished by mohair paint roller, to interior or exterior new or old concrete. It cures overnight, at normal room temperatures, to a dense, slip-resistant, nonshrinking topping of excellent impact and abrasion resistance according to the developer. It is further said to have a proved resistance to the corrosive action of over 70 chemicals encountered in manufacturing and food processing plants.

Manufacturer also recommends the compound for repairing cracks and grouting the joints of acid-proof brick and quarry tile floors, and as a covering on adjoining vertical surfaces, curbing, acid trenches and solvent or alkali sewers. Available in cement grey and tile red in 10 gal., 2 gal., and 6 qt units. Each consists of equal number of units of base compound and catalyst, premeasured at the factory, for quick and easy mixing and placing at the job site.—A. C. Horn Companies, 2133 85th St., North Bergen, N.J.

Lightweight, ready-to-use screed

Designed primarily for bridge and overpass road construction, the Yaun Screed packs and settles concrete around steel reinforcing rods, shaping the crown and slope of the roadway at the same time.



The Yaun Screeds are economical and ready for immediate use since they are built to meet the customer's own specifications, both in over-all length and thickness.—Yaun Manufacturing Co., Inc., 2120 North Third, Baton Rouge, La.

Color added to truck mixer

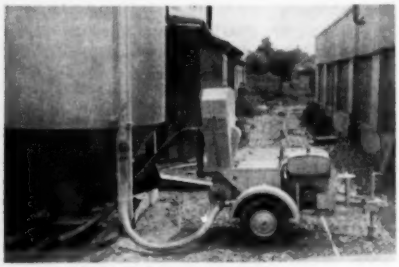
Colorfull, a new coloring product for ready-mixed concrete, is designed to be added to the contents of a mixer truck, at the rate of 1 bag per cu yd of material. The resulting product, a uniform ready-mixed concrete,

contains color through the entire mass. The color is said to be permanent, nonfading and impossible to wear through.

Ten colors are available: slate, tan, red, coral, light red, ebony, Navajo blue, gold, green, and adobe. Special colors are available in projects of 100 cu yd or more. The average cost per sq ft for most colors is slightly more than ordinary concrete according to manufacturer.—Admixtures, Inc., Pasadena, Calif.

Air caddy moves cement

Photograph below shows the Ripco Air System being used at the Trans-Mix plant in Easton, Pa., to move cement from storage



bin to batch bin. The unit is also used to unload hopper cars and fill the storage bin. There is no spillage and no need for an under-track receiving pit according to manufacturer.

The Ripco unit, either gasoline or electrically powered, reportedly can unload hoppers at 1000 lb per min with one man.—Ripco Air Systems, Oxford, Pa.

Polysulfide modified adhesive mortar fills dip in California bridge roadway

How to fill a rut or dip in concrete roadways quickly yet with watertight permanence, has been demonstrated recently by a remedial project on the Richmond-San Rafael Bridge in California. A dip in the bridge roadway, caused by failure of mortar surfacing, was a potential threat to highway safety. Up to ¾ in. deep and 10 ft wide, the dip stretched 36 ft across several lanes.

Since only a ¾ in. overlay was required to fill the dip, the engineers decided to employ a polysulfide modified epoxy mortar—a mixture of the adhesive and aggregate. Thiokol LP 3 liquid epoxy resins and DMP 30 were major ingredients in the adhesive composition developed by the Division of Highways. After these ingredients were properly mixed, enough white filler was combined to give the finished mortar a concrete-like appearance to match the rest of the roadway. The adhesive was poured into a mortar box and combined with aggregate in a 1:15 ratio.

Sandblasting was required to clean off loose particles and oil film to give the mortar a good surface to which it could bond. Actual application of the mortar was similar to applying concrete. As the void was filled with mortar, a screed was worked over the material to smooth the surface. Since little solvent was used in the compound, shrinkage was not a problem. Sand was sprinkled over the application, and a wood float was used to smooth the material.

Pot life of this chemically cured mortar is less than an hour. Curing time varies with the temperature; time is reduced as the temperature is increased to 250 F. At 80 F the mortar would be sufficiently hard to take traffic in about 12 hr. By applying heat the curing time could be reduced to 1 hr. The highway engineers compromised by heating the applied mortar for about a ½ hr and leaving the lanes closed overnight. Propane burners were used to apply the heat. Actually the road could have been opened within 4 hr without damage to the resurfaced area. —Thiokol Chemical Corp., Trenton, N.J.

Vacmobile cleans while you drill

The lightweight, portable, heavy-duty Pullman Vacmobile recovers water, dust, and tiny particles of concrete, plaster and stone from all types of drilling applications. Manufacturer claims the unit is easily adaptable for both dry and wet drilling, indoor and outdoor jobs, on concrete, marble, or stone. No outside bag to snag or tear, it features the "Never-Clog" secondary filter.

Available in 3 models with various horsepower; 2 models can be converted to a blower. Other specifications include: 115 volts, zero to 60 cycles, AC-DC, odd voltages available on request. — Pullman Vacuum Cleaner Corp., 25 Buick St., Boston 15, Mass.

Safe power fastening tool

Developed by Hilti's research department, the DX 100-L fastening tool was designed for safe, fast, economical fastenings into dense materials such as concrete, concrete or cinderblock, and structural steel. It provides guaranteed safety features that eliminate the hazards of ricochet, fish-hooking, and explosive spall encountered with "gun-type" tools according to Hilti engineers.

The chief limitation of the tool compared to conventional stud guns is that the maximum fastener size it can drive is a ¼ in.-20 stud. This prevents its use in the heaviest fastening applications. However, in the majority of cases it works effectively according to manufacturer. It has been found to be not only safe, but also fast and convenient to use in hard-to-get-at installations.

Hilti conic studs and pins used in the unit are made of special alloys, rustproofed and specially tapered for maximum holding power. Literature available.—Hilti, Inc., 73 Southfield Ave., Stamford, Conn.

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Literature Available

Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital letters. They may be requested directly from the manufacturers listed below.

HORNCURE CONCRETE CURING COMPOUNDS—Catalogs covering concrete curing compounds, developed by the A. C. Horn Research Laboratories, describe the 100 percent Resin Base Curing Compound, the White Pigmented Curing Compound, and the Wax Resin Base Curing Compound, together with their advantages, uses, specifications, and quantities required.—A. C. Horn Companies, Division of Sun Chemical Corp., 2133 85th St., North Bergen, N.J.

THE ULTIMATE IN VIBRATIONS—Folder describes the Oswalt vibrator assembly featuring their vibrating shaft with replaceable sheave, providing a method of maintaining exacting vibrations per cycle in machine operation with the replacement of a few easily accessible parts of the entire vibrating shaft. Parts can be purchased as separate units or as a whole assembly.—Oswalt Engineering Service Corp., 1335 Circle Ave., Forest Park, Ill.

CONCRETE FOR STADIUMS AND AUDITORIUMS (MBR No. 12)—Concreting problems encountered and solved in the construction of 16 outstanding stadium and auditorium projects in this country and abroad are explained in this 20-page publication. The role played by Pozzolith in the projects is stressed.—The Master Builders Co., Cleveland 18, Ohio

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